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# Development of Geotechnical Resistance Factors and Downdrag Load Factors for LRFD Foundation Strength Limit State Design

## Reference Manual

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# NOTICE

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# Forward

With the advent of the AASHTO Load and Resistance Factor (LRFD) Bridge Design Specifications in 1992, there has been considerable focus on the geotechnical aspects of those specifications, since most geotechnical engineers are unfamiliar with LRFD concepts. This is especially true regarding the quantification of the level of safety needed for design. Up to the time of writing of this report, the geotechnical profession has typically used safety factors within an allowable stress design (ASD) framework (also termed working stress design, or WSD). For those agencies that use Load Factor Design (LFD), the safety factors for the foundation design are used in combination with factored loads in accordance with the AASHTO Standard Specifications for Highway Bridges (2002).

The adaptation of geotechnical design and the associated safety factors to what would become the first edition of the AASHTO LRFD Bridge Design Specifications began in earnest with the publication of the results of NCHRP Project 24-4 as NCHRP Report 343 (Barker, et al., 1991). The details of the calibrations they conducted are provided in an unpublished Appendix to that report (Appendix A). This is the primary source of resistance factors for foundation design as currently published in AASHTO (2004). Since that report was published, changes have occurred in the specifications regarding load factors and design methodology that have required re-evaluation of the resistance factors. Furthermore, new studies have been or are being conducted that are yet to be implemented in the LRFD specifications.

In 2002, the AASHTO Bridge Subcommittee initiated an effort, with the help of the Federal Highway Administration (FHWA), to rewrite the foundation design sections of the AASHTO LRFD Bridge Design Specifications due to the many difficulties geotechnical engineers were having with implementation of the LRFD specifications. This report on resistance factor and downdrag load factor development was written to assist in that effort, providing the necessary background and justification for the recommended resistance factors (and downdrag load factor).

It should be recognized that this report is not a state-of-the-art summary of research conducted to develop LRFD for geotechnical design. This report has been written to support a state-of-practice document (the AASHTO LRFD specifications). Therefore, the focus of this report is the state-of-practice, and the implementation of completed research that has been targeted specifically to the AASHTO LRFD specifications and that is ready for state-of-practice implementation. This report is a mixture of mathematical rigor, logic, and policy level decision documentation, in which the purpose is to provide an historical background regarding how the AASHTO LRFD foundation resistance factors came to be, including their basis, to also provide a summary of new calibration work that needs to be incorporated in the specifications, and to specifically describe the logic used to establish the final resistance factors recommended for the next edition of the AASHTO LRFD specifications (i.e., year 2005).

The input provided by the Federal Highway Administration (FHWA) representatives, the FHWA Technical Working Group, and their consultants, including Jerry DiMaggio (FHWA), Firas Ibrahim (FHWA), Jawdat Siddiqi (OHDOT), Larry Jones (FLDOT), Susan Hida (CALTRANS), Rebecca Burns (PENNDOT), Chis Benda (VTDOT), Jugesh Kapur (WSDOT), Raymond Hartle (Michael Baker Corp), Robert Kimmerling (PanGeo, Inc.), and George Goble (Goble LLC) is gratefully acknowledged. The technical input provided by two anonymous reviewers of this report is also greatly appreciated. Their input significantly improved this report.

# Executive Summary

One of the most difficult hindrances to implementation of the geotechnical aspects of the LRFD Bridge Design Specifications has been acceptance of the geotechnical load and resistance factors. Full acceptance by the geotechnical profession has not been gained for the load and resistance factors provided in the first edition of the AASHTO LRFD Bridge Design Specifications (1992). However, recent research specifically focused on load and resistance factor development has been conducted and is awaiting full implementation. Hence, both the basis of the current load and resistance factors, and the logic and background needed to understand and accept changes in those factors based on new information, must be explained.

The purpose of this report is to summarize the historical development of the resistance factors needed for geotechnical foundation design, and to recommend how to specifically implement the recent developments in resistance factors for geotechnical foundation design. In addition, recommendations regarding the load factor for downdrag loads, based on statistical analysis of available load test data and reliability theory, and in consideration of historical design practice, are provided. The scope of this report is limited to shallow and deep foundation design.

It should be recognized that this report is not a state-of-the-art summary of research conducted to develop LRFD for geotechnical design. The focus of this report is the state-of-practice, and the implementation of completed research that has been targeted specifically to the AASHTO LRFD specifications and that is ready for state-of-practice implementation.

The source of the resistance factors in the current AASHTO LRFD Design specifications (AASHTO 2004), Section 10, is NCHRP Report 343 by Barker, et al. (1991). The details of the calibrations conducted are included in an unpublished appendix to that report (Appendix A). Resistance factors were developed using statistical data and reliability theory (if such data was available), and calibration by fitting to Allowable Stress Design (ASD). In many cases, results from calibration by fitting to ASD influenced the final value selected for the resistance factors reported by Barker, et al. (1991) and incorporated in the LRFD specifications (AASHTO 2004) for foundation design. It must be recognized that calibration by fitting to ASD does not meet the objective of providing a consistent level of safety in design. However, calibration by fitting provides the strongest link to previous ASD practice and provides a designer's perspective to reliability analysis results.

The load factors available at the time the original resistance factors were developed are different from the load factors specified in the current AASHTO LRFD specifications (AASHTO 2004). This change has resulted in a minor, but not insignificant, difference in the resistance factors needed to be consistent with previous ASD practice, and for those cases where adequate statistical data were available, to be consistent with the desired margin of safety. Minor changes in load statistics have also occurred since the original work was conducted, and in some cases, the design methodologies recommended in the AASHTO specifications have changed or are proposed to be changed, resulting in the need for reassessment of the current resistance factors.

New studies have been or are being conducted that are yet to be implemented in the LRFD specifications. At the time of writing this report, the only calibration research results yet to be implemented are the results of NCHRP Project 24-17 on deep foundations, published as NCHRP Report 507 (Paikowsky, et al., 2004). Therefore, only the results provided by Barker, et al. (1991) and Paikowsky, et al. (2004) are addressed in detail herein.

Specific analyses of the resistance factors for the various foundation types (footings, shafts, and piles) are provided. Reliability analyses using the Monte Carlo method were also conducted by the writer, for those cases where statistical data were available, to verify the previous results and to extend them to the new load factors, also addressing any other changes in the input data that may have occurred since the original work was accomplished. For pile and shaft foundations, the results from Paikowsky, et al. (2004) were evaluated in light of this previous work and historical design practice to develop resistance factor recommendations. Recommendations are provided regarding resistance factors that should be included in the AASHTO LRFD Specifications. These recommendations are based on the theoretical results from reliability analyses, with consideration of past design practice as a benchmark.

Statistical data from Paikowsky, et al. (2004), were used to assess the load factors needed to account for downdrag. For both piles and shafts, the uncertainty in the downdrag load was assumed to be represented by the bias and COV of the method used to estimate the skin friction contributing to downdrag. Reliability analyses using the Monte Carlo method were conducted to make this assessment. The results indicate that the magnitude of the load factor for downdrag depends on the foundation type and the magnitude of the downdrag load relative to the other loads applied to the foundation, and ranged from 1.0 to 1.6.

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# 1. Introduction

Since the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor (LRFD) Bridge Design Specifications first became available in 1992, agencies that use the AASHTO specifications have had significant difficulty with implementation of the geotechnical sections of the specifications. Particularly difficult has been how to relate the factors of safety (FS) that have been in use for many years to the load and resistance factors required in the LRFD specifications. For the discipline of geotechnical engineering, engineering judgment can play a significant role in the selection of FS values for design, based on an intuitive assessment of the effect the variability at a given site, and the effect the design assumptions used should have on the final FS selection. It is therefore important for geotechnical engineers who use the LRFD specifications to understand the basis of the load and resistance factors provided in the specifications, and how those factors relate to the FS values that have been, and still are being, used in geotechnical design practice.

A key objective of LRFD is to establish a level of safety that is consistent across various limit states of a given type (e.g., strength limit states) to design the various components that make up a structure (e.g., foundations, columns, superstructure, retaining walls, etc.). To accomplish this, statistical data used as input for reliability analyses must be conducted. These reliability analyses allow one to theoretically calculate the magnitude of FS values, or in LRFD terms, load and resistance factors ($\gamma$ and $\varphi$, respectively), needed to obtain the desired (target) level of safety needed. In past, and current, allowable stress design practice (ASD; or working stress design – WSD), the FS was based on engineering judgment and long-term experience. If failures started occurring when using the selected FS values, increases in the FS were made, again based on judgment, to reduce the recurrence of performance problems to an acceptable level. If no failures occurred, FS values were in general not reduced to get closer to the level of safety desired (i.e., to just above the level where an unacceptable number of failures begins to occur), causing FS values to tend to be overly conservative. Therefore, while not theoretically rigorous, the development of FS values has at least, based on judgment and long-term experience, considered some desired level of safety, though that level of safety may not be consistent across limit states and may not be at the target level for LRFD structural and geotechnical design. This is a key difference between ASD and LRFD.

Considering the theoretical nature of reliability analyses, should those who make the final decisions on the load and resistance factor values fully rely on the reliability analysis results? Alternatively, should the load and resistance factors determined through calibration by fitting to ASD FS values (i.e., determining the load and resistance factor combination that yields the same numerical value as the ASD FS) be allowed to influence the final load and resistance factor selection, recognizing that the ASD FS may not have a level of reliability that is consistent with the target reliability level? For example, should the resistance factor be made more conservative based on reliability analysis results when considerable long-term successful experience indicates that the existing degree of conservatism is adequate? The answer to this question depends primarily upon the adequacy (e.g., quality and quantity) of the statistical data used as input into the reliability analyses, and how well that data characterizes reality.

The purpose of this report is to summarize the historical development of the resistance factors needed for geotechnical foundation design, and to recommend modifications to these resistance factors based on the recent developments. The source of the resistance factors in the current AASHTO LRFD Design Specifications (AASHTO 2004), Section 10, is NCHRP Report 343 by

Barker, et al. (1991). The details of the calibrations conducted are included in an unpublished appendix to that report (Appendix A). Therefore, Barker, et al. and the AASHTO LRFD Bridge Design Specifications are the primary sources of historical resistance factor development information. At the time of writing this report, the only calibration research results available for use but yet to be implemented are the results of NCHRP Project 24-17 on deep foundations, published as NCHRP Report 507 (Paikowsky, et al., 2004). Therefore, only the calibration results provided through Barker, et al. (1991) and Paikowsky, et al. (2004) are addressed in detail herein.

The focus of this report is the state-of-practice, and the implementation of completed research that has been targeted specifically to the AASHTO LRFD specifications and that is ready for state-of-practice implementation. The scope of this report is limited to the development of resistance factors for strength limit state design of shallow and deep foundations. Load factors recommended in the current AASHTO LRFD Bridge Design Specifications (AASHTO 2004) are used to establish the recommended resistance factors. In addition, recommendations regarding the load factor for downdrag loads, based on statistical analysis of available load test data and reliability theory, and in consideration of historical design practice, are provided.

# 2. Resistance Factor Development Background

The two most commonly used approaches to develop resistance factors for LRFD are:

a) calibration by fitting to ASD, and
b) calibration using reliability theory.

Calibration by fitting is usually done after there has been a fundamental change in either design philosophy or design specification format (e.g., ASD to LRFD format) and detailed statistical data have not yet been gathered. Calibration by fitting to ASD is also used as a benchmark when considering the adjustment of LRFD resistance factors based on calibration using reliability theory. In calibration by fitting, the resistance factors in the new (LRFD) specifications are adjusted such that designs similar to the old (ASD) specifications are obtained. Calibration using reliability theory involves the estimation of the reliability inherent in current design methods, selecting a target reliability based on the margin of safety implied in current designs, and then determining load and resistance factors consistent with the selected target reliability.

Barker, et al. (1991) developed resistance factors (termed performance factors in their report) using statistical data, for those cases where such data were available, and calibration by fitting to Allowable Stress Design (ASD). In many cases, calibration by fitting to ASD controlled or at least influenced the final value selected for the resistance factor.

The source of the load factors used by Barker, et al. (1991) was the AASHTO Load Factor Design (LFD) specifications, which are contained in the AASHTO Standard Specifications for Highway Bridges (AASHTO 2002). It is important to note that the load factors used in NCHRP 343 were different from those in the current AASHTO LRFD (2004) specifications. The pertinent LFD and LRFD load factors are summarized in Table 1. The load factors used affect both calibration by fitting to ASD and calibration using reliability theory. Since the load factors have changed since the original work by Barker, et al. (1991), adjustment of the resistance factors to reflect the current load factors is needed.

For both calibration by fitting and reliability theory, the calibration conducted previously by others (Barker, et al., 1991; Paikowsky, et al., 2004) was focused on Strength I, the most common strength limit state, focusing only on dead load and live load. While some bridges are designed using

Strength IV, this load group typically applies only to very long span bridges (typically steel). Therefore Strength I was selected for calibration purposes. Within this load group, dead load and live load are the most common loads considered for design.

**Table 1. LFD and LRFD load factors used in resistance factor calibration.**

| | LFD Load Factors (Used in the NCHRP 343 Calibrations) | 2004 LRFD Load Factors |
|---|---|---|
| Dead load, $\gamma_{DL}$ | 1.30 | 1.25 |
| Live load, $\gamma_{LL}$ | 2.17 | 1.75 |

## 2.1 Calibration by Fitting

Calibration by fitting to ASD is conducted for the case where two load sources are considered using the following equation:

$$\varphi = \frac{\gamma_{DL} \, {}^{DL}\!/_{LL} + \gamma_{LL}}{\left({}^{DL}\!/_{LL} + 1\right) FS} \tag{1}$$

where, φ the resistance factor, $\gamma_{DL}$ is the load factor for the dead load, $\gamma_{LL}$ is the load factor for the live load, DL/LL is the dead load to live load ratio, and FS is the allowable stress design factor of safety. As can be seen from this equation, there is no consideration of the statistical parameters associated with the loads and the resistance, and therefore no consideration of the margin of safety inherent in the FS.

If using the current LRFD load factors (AASHTO 2004),

$$\varphi = \frac{1.25 \, {}^{DL}\!/_{LL} + 1.75}{\left({}^{DL}\!/_{LL} + 1\right) FS} \tag{2}$$

If DL/LL = 3.0, then $\varphi = \frac{1.25(3.0) + 1.75}{(3.0 + 1) FS} = \frac{1.375}{FS}$

If using the LFD load factors, such as used by Barker, et al. (1991), then

$$\varphi = \frac{1.30 \, {}^{DL}\!/_{LL} + 2.17}{\left({}^{DL}\!/_{LL} + 1\right) FS} \tag{3}$$

If DL/LL = 3.0, then

$$\varphi = \frac{1.30(3.0) + 2.17}{(3.0 + 1) FS} = \frac{1.518}{FS}$$

Note that these equations simply provide a way to obtain an average value for the load factor considering the use of two different load factors from two different sources, and considering the relative magnitudes of each load. In the most basic terms, the ASD FS is simply the average load factor divided by the resistance factor. There is no consideration of the actual bias or variability of the load or resistance prediction methods when "calibration" by fitting to ASD is used, nor is there any consideration to the probability of failure, $P_f$. All that is being done here is to calculate the magnitude of a resistance factor, for a given set of load factors, that when combined with the load factors, provides the same magnitude of FS as is currently used for ASD. Therefore, whatever margin of safety was implied by the ASD safety factor, the load and resistance factor combination that results from this type of analysis will have the same unknown margin of safety. Without conducting some type of reliability analysis based on statistical data for the loads and resistances under consideration, or without some type of quantification of the number of failures relative to the number of successes when the safety factor in question is used in conjunction with a given design procedure, the margin of safety implied by the safety factor, and the resistance factors derived using this approach, will be unknown.

As can be observed from Equations 1, 2, and 3, the resistance factor is a function of the DL/LL ratio. The DL/LL of 3.0 is a typical number that has been used in past calibrations (Barker, et al., 1991) though a variety of DL/LL ratios have been analyzed to assess the effect of this parameter. The resistance factor calibrated by fitting to ASD does vary with this ratio. Past calibration recommendations (e.g., Barker, et al., 1991) have not advocated a resistance factor that varies with the relative magnitude of the various loads in the load group. Furthermore, when evaluating the same limit state using reliability theory (discussed later), it has been found in general that the resulting resistance factor is not all that sensitive to the DL/LL ratio. The resistance statistics tend to overwhelm the calibration, because the coefficient of variation (COV) for the resistance is so much larger than the COV for these loads. This causes the sensitivity of the resistance factor to the DL/LL ratio when doing calibration by fitting to ASD to be exaggerated when compared to resistance factors determined from reliability theory. The DL/LL ratio will be different for each structure, depending on span length and other factors. It would be too complicated to provide resistance factors as a function of DL/LL ratio, or possibly other loads within the group in question. As discussed in more detail later, a DL/LL ratio of 3.0 is used to be consistent with the conclusions made in previous work (e.g., Barker, et al., 1991) and to provide a direct comparison to the work conducted for the original development of resistance factors for foundations regarding proposed resistance factor changes.

**Table 2. Resistance factors determined from equations 2 and 3 for a DL/LL ratio of 3.0.**

| FS | Resistance Factor | |
|---|---|---|
| | Using LFD Load Factors (Equation 3; Barker, et al., 1991) | Using LRFD Load Factors (Equation 2; AASHTO 2004) |
| 1.5 | 1.0 | 0.92 |
| 1.8 | 0.84 | 0.76 |
| 1.9 | 0.80 | 0.72 |
| 2.0 | 0.76 | 0.69 |
| 2.25 | 0.67 | 0.61 |
| 2.5 | 0.61 | 0.55 |
| 2.75 | 0.55 | 0.50 |
| 3.0 | 0.51 | 0.46 |
| 3.5 | 0.43 | 0.39 |
| 4.0 | 0.38 | 0.34 |

The resistance factors corresponding to various ASD factors of safety as determined from Equations 2 and 3 are summarized in Table 2. As can be seen from Table 2, for a given FS, the resistance factors based on the load factors used by Barker, et al. (1991) are higher than those obtained using the AASHTO (2004) load factors. Thus, to account for the differences in the load factors derived from calibration by fitting by Barker, et al. and the AASHTO (2004) LRFD load factors, the Barker, et al. resistance factors (for calibration by fitting to ASD) should be decreased by approximately 10 percent (i.e., 1.375/1.518 = 0.91).

## 2.2 Calibration Using Reliability Theory

Reliability theory is used to determine the likelihood that failure (i.e., as represented by the probability of failure, $P_f$) will occur for a given structural component, in consideration of the selected load and resistance factor combination (or safety factor), the design method selected, and any other conditions that may affect the performance of the component. For these analyses, failure is simply defined as the case where the loads applied to a given structure or structure component (in this case, a foundation unit) are greater than the available resistance. Hence,

$$\sum \gamma_i Q_{ni} \leq \varphi R_n \qquad (4)$$

where, $\gamma_i$ is a load factor applicable to a specific load type $Q_{ni}$; the summation of $\gamma_i Q_{ni}$ terms is the total factored load for the load group applicable to the limit state being considered; $\varphi$ is the resistance factor; and $R_n$ is the nominal unfactored (design) resistance available (either ultimate or the resistance available at a given deformation).

The loads ($Q_{ni}$) and resistance ($R_n$) are treated as random variables. A random variable is a parameter that can take different values that are not predictable. However, the distribution of the frequency of occurrence of those random values can be characterized using a distribution function (e.g., normal, lognormal, etc.) and statistical parameters such as the mean and standard deviation.

The objective of calibration using reliability theory is to estimate the combination of load and resistance factors that separates the load and resistance values enough so that the situation where the applied loads, Q, are greater than the available resistance, R, occurs only rarely (i.e., the probability of failure is acceptably small). Q and R are random variables representing the load and resistance, respectively. This is illustrated in Figure 1. The probability of failure, $P_f$, is represented in the reliability analysis by the reliability index $\beta$. The reliability index $\beta$ represents the distance measured in standard deviations between the mean safety margin and the failure limit. The load and resistance factors are set such that the probability of failure (i.e., failure occurs when R-Q is less than zero) as determined from the reliability analysis, is acceptably small.

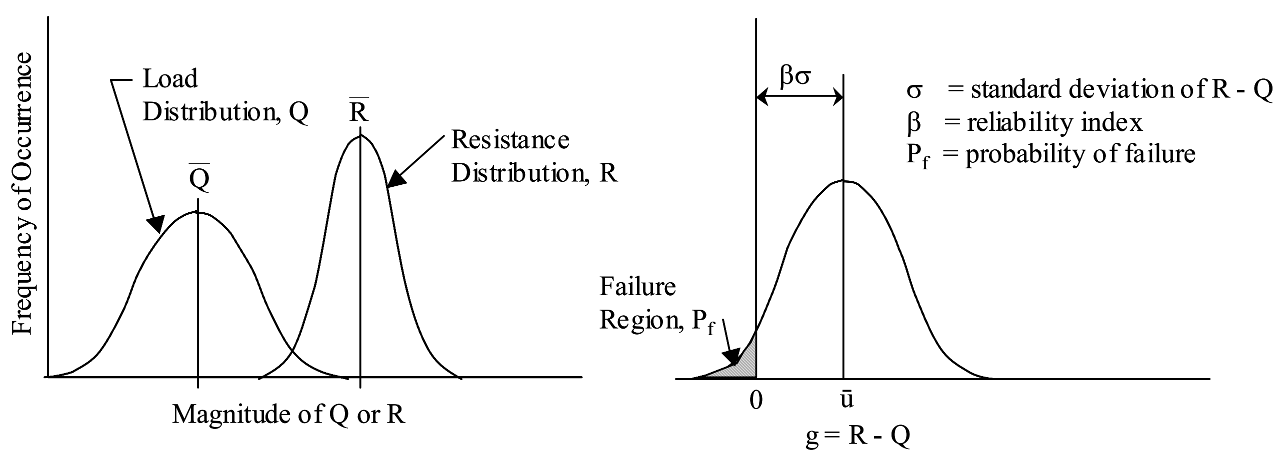


**Figure 1. Probability of failure and reliability index (adapted from Withiam, et al. 1998).**

The general process used to perform calibration using reliability theory is to:

- Gather the data needed to statistically characterize the key load and resistance random variables, developing parameters such as the mean, COV, and distribution type (e.g., normal, lognormal),
- Estimate the reliability inherent in current design methods,
- Select a target reliability based on the margin of safety implied in current designs, considering the need for consistency with the reliability used in the development of the rest of the AASHTO LRFD specifications, and considering levels of reliability for geotechnical design as reported in the literature, and
- Determine load and resistance factors consistent with the selected target reliability.

This process was used by Barker, et al. (1991) and Paikowsky, et al. (2004), and has been considered in this report in the development of resistance factors.

There are several techniques that can be used to conduct the actual reliability analysis. These methods vary in their degree of sophistication and theoretical accuracy. These techniques are briefly described in the paragraphs that follow.

The simplest technique is Mean Value First Order Second Moment Method (MVFOSM). In this approach, the random variables are represented by their first two moments, the mean, $\mu$, and standard deviation, $\sigma$, (i.e., statistical parameters that characterize the average value of all the data, and the spread in the data from the mean, respectively). Note that the COV is simply the standard deviation normalized by the mean, or $\sigma/\mu$. Hence the COV can be used in lieu of the standard deviation to represent the spread in the data. The limit state function "g" is linearized at the mean values (i.e., $\mu$) of the random variables, rather than at a point on the failure surface (i.e., where the load Q is equal to the resistance R), hence the term "mean value." The mean value and standard deviation of "g" are determined from a Taylor series expansion in which only the first order terms are considered, hence the term "first order." This linearization and the neglecting of the higher order terms can be sources of error, especially for nonlinear limit state functions. Paikowsky, et al. (2004) demonstrated that the difference between the results obtained using MVFOSM and the more advanced methods described below is typically around 10 percent for linear limit state functions.

Advanced methods are available that do not rely on the mathematical simplifications needed to perform MVFOSM. These advanced methods are described by Hasofer and Lind (1974) and Rackwitz and Fiessler (1978). These advanced methods consider the mean, standard deviation, and distribution function (e.g., normal, lognormal, etc.) for the data obtained to characterize each random variable.

More recently, another advanced method, Monte Carlo simulation, has been used for performing these reliability analyses. Allen et al. (in press) and Nowak and Collins (2000) have shown that all of these advanced methods should produce similar results. Monte Carlo simulations were conducted by the writer using the statistics reported by Barker, et al. (1991) to verify the resistance factors reported in NCHRP Report 343 and to extend the reliability theory results to the LRFD load factors (see Table 1). Detailed information on this calibration process and the tools used to conduct the reliability analysis (i.e., MVFOSM, advanced methods such as Rackwitz and Fiessler (1978) and Monte Carlo simulation) are provided by Allen, et al. (in press).

### 2.3 Calibration Approach Used by Barker, et al. (1991)

In general, statistical data gathered for use in the reliability theory calibrations by Barker, et al. (1991) were obtained from the published literature. The statistical data can be categorized as follows:

- Statistics representative of inherent spatial variability
- Statistics representative of systematic error
- Statistics representative of design model error

Spatial variability is related to the variability of the measured input parameters over a distance, area, or volume of the material being evaluated. Soil and rock properties in particular are known to vary from point to point, causing the measurement of a given property at a point to have a higher variability/uncertainty than the average of a number of measurements taken at various points in the soil or rock deposit surrounding the foundation element to be designed. Systematic error is primarily focused on the repeatability of the tests used to measure the property or input value under consideration. Design model error is focused on the ability of the design model itself to accurately estimate the resistance, i.e., how well does theory match reality? For geotechnical design, and the theories or empiricism used to develop geotechnical design methods, design model uncertainty and bias can be significant due to the nonlinear nature of soil properties, their tendency to be highly variable, and the assumptions needed to make the design model feasible to use.

These three main sources of uncertainty must be somehow combined to obtain the statistics needed as input for the reliability analysis. Barker, et al. (1991) took a first order approach to combining these three sources of uncertainty to obtain the final statistics used as input in the reliability analyses they conducted. This approach (see Equations 5 and 6) assumes that all three sources of uncertainty are completely independent.

$$\lambda_R = \lambda_1 \times \lambda_2 \times \lambda_3 \tag{5}$$

$$COV_R = \sqrt{COV_1^2 + COV_2^2 + COV_3^2} \tag{6}$$

where, $\lambda_R$ is the combined bias for all the sources of uncertainty affecting the resistance prediction, bias is the mean of the measured/predicted values in the dataset, $\lambda_1$, $\lambda_2$, and $\lambda_3$ are the bias values for the individual sources of uncertainty, $COV_R$ is the combined coefficient of variation for all the sources of uncertainty affecting the resistance prediction, and $COV_1$, $COV_2$, and $COV_3$ are the COV values for the individual sources of uncertainty.

Model uncertainty is typically obtained through resistance measurements obtained from full scale or model scale foundation units or structures, where nominal (predicted) values of the resistance are obtained using measured input parameters (e.g., soil shear strength, soil unit weight, etc.) and the theoretical/empirical design model being evaluated, with the bias and COV for the design model calculated using the ratio of the measured to predicted resistance. Because of the way full or model scale resistance data is obtained, the variability of the measured resistance likely already contains at least some spatial and systematic error. Therefore, combining these three sources of error together is likely conservative, especially for full scale resistance data. For model scale resistance data, the environment and variables are much more carefully controlled, and therefore the model scale resistance data may not address all sources of error. In any case, Barker, et al. (1991) combined these sources of error to develop the final statistics used as input in the reliability analyses they conducted. More recent studies (e.g., Paikowsky, et al., 2004 – see Section 2.4) have assumed that the measurements from full scale field installations adequately addressed all these sources of error, and hence there was no need to add the error due to systematic error or spatial uncertainty to the error determined from the field measurements.

Barker, et al. (1991) only provided summary statistics (i.e., mean and COV) obtained from the literature. A detailed analysis demonstrating how well the given data fit the assumed distribution was not provided, and in most cases the distribution assumed (e.g., normal, lognormal, etc.) was not provided. However, it appears that they assumed the resistance statistics were lognormal, which is consistent with this writer's experience and the experience of others (Paikowsky, et al. 2004) regarding foundation resistance. They assumed that the dead load and live load statistical distributions are normal, which is consistent with Nowak (1999).

For footings, the primary source of statistical data was from small scale model test results provided in the literature, though for some of the data sources reported, the overall size of the footings was not specifically described. The size of the database used was not reported. For drilled shafts, Barker, et al. (1991) report that the primary source of statistical data was from load test results provided by Reese and O'Neill (1988) and Horvath and Kenney (1979), for a total of 76 load test case histories. For piles in clay, the primary source of statistical data was from Sidi (1986). The number of pile load tests was not specifically reported, but was described as numerous. The specific source of statistical data for piles in sand, at least with regard to model error, was reported as from Robertson, et al. (1988) and Horvitz et al. (1981) load test data for the CPT method. For the Meyerhof SPT method in sand, Barker, et al. considered the bias and COV of the SPT test to be adequate for the development of model error statistics for this method.

Barker, et al. (1991) used load statistics from Grouni and Nowak (1984) and the LFD load factors in use at that time in the AASHTO Standard Specifications for Highway Bridges. The load statistics used are as shown in Table 3. For the analyses they conducted, they evaluated a range of DL/LL ratios, but settled on a ratio of 3.0 for the final analyses. Resistance factors were developed by Barker, et al. for two span lengths, 60 and 160 ft. Regarding span length, a 160 ft span corresponds reasonably well to the bias value currently used to represent the live load (i.e., 1.15 for the combination of static and dynamic live load – see Section 2.4). For convenience, this span length was targeted for the comparisons provided herein.

**Table 3. Load statistics and factors used by Barker, et al. (1991).**

| Load Type | Bias | Coefficient of Variation | Load Factor Used |
|---|---|---|---|
| Dead load | $\lambda_D = 1.05$ | $COV_{QD} = 0.09$ | $\gamma_{DL} = 1.3$ |
| Live load | $\lambda_L = 1.05$ to 1.22, depending on span length | $COV_{QL} = 0.11$ | $\gamma_{LL} = 2.17$ |

Barker, et al. (1991) used a combination of calibration by fitting to ASD, and reliability theory. When reliability theory was used, they used MVFOSM for preliminary analyses and advanced methods (e.g., Hasofer and Lind, 1974) for final analyses.

Once resistance factors equivalent to ASD FS values were determined through calibration by fitting, Barker, et al. (1991) used MVFOSM (as well as the advanced method for selected limit states) to assess, using the available statistical data for each random variable the level of safety (β) implied by the ASD FS value for the limit state under consideration. In their analyses, the load(s), Q, and the resistance R, were treated as the random variables, allowing them to use a simple, linear limit state function $g = R - Q$. The following MVFOSM equation was used for this purpose:

$$\beta = \frac{LN\left[\frac{\lambda_R FS(Q_D/Q_L+1)}{\lambda_D Q_D/Q_L+\lambda_L}\sqrt{\frac{1+COV_R^2+COV_{QD}^2+COV_{QL}^2}{1+COV_R^2}}\right]}{\sqrt{LN\left[\left(1+COV_R^2\right)\left(1+COV_{QD}^2+COV_{QL}^2\right)\right]}} \tag{7}$$

where,
$\beta$ = the reliability index
$Q_D$ = nominal value of the dead load
$Q_L$ = nominal value of the live load ($Q_D/Q_L$ is the dead load to live load ratio)
$\lambda_R$ = mean of the bias values (measured/predicted) for the resistance
$\lambda_D$ = the mean of the bias values (measured/predicted) for the dead load
$\lambda_{LL}$ = the mean of the bias values (measured/predicted) for the live load
$FS$ = the factor of safety used in ASD
$COV_{QD}$ = the coefficient of variation of the bias values for the dead load
$COV_{QL}$ = the coefficient of variation of the bias values for the live load
$COV_R$ = the coefficient of variation of the bias values for the resistance

In general, resistance factors for bridge and other structural design have been derived to produce a β of 3.5 (i.e., an approximate probability of failure, $P_f$, of 1 in 5,000). However, Barker, et al. (1991) found that, based on the results of the analyses they conducted using Equation 7 as well as using the advanced method, the β value implied by the ASD FS values used in past geotechnical design practice for foundations was typically less than this, especially for the more redundant foundation systems such as pile groups. Specifically, they determined that β varied from 1.3 to 4.5 for bearing capacity of footings on sand determined using Rational Theory, with the low values of β corresponding to footings in which the foundation soil friction angle was high (i.e., $39^o$ or more). For footings on clay, β varied from 2.7 to 5.7. For semi-empirical methods of estimating footing bearing capacity, β was approximately 4.2 for the SPT method and 3.3 for the CPT method. For shafts, for bearing, β ranged from 2.0 to 3.7 using MVFOSM and 2.0 to 4.3 using the advanced method. For pile foundations, for bearing, β ranged from 1.6 to 3.1 using MVFOSM and 1.6 to 3.3 using the advanced method. They concluded that target β values of 3.5 for non-redundant systems (e.g., footings), 2.5 to 3.0 for drilled shafts, and 2.0 to 2.5 ($P_f$ of approximately 1 in 100) for highly redundant systems, such as pile groups, should be used. Note that in more recent work, Zhang, et al. (2001) and Paikowsky, et al. (2004) have also concluded that for redundant foundation systems such as pile groups, a β of approximately 2.3 is reasonable to target when considering the resistance needed for a single pile or shaft located within a group of significant size.

Once the target β value was determined for the limit state under consideration, the advanced reliability theory method was used by Barker, et al. (1991), to calculate the resistance factor needed to obtain the desired margin of safety as represented by the target β value. In many cases they used, or were at least influenced by, the resistance factors obtained from calibration by fitting to ASD when making a final resistance factor selection, in spite of what the reliability analyses implied the resistance factors should be. This approach considers that if the factors of safety used in past practice have resulted in consistently successful designs, one will at least maintain that degree of success at the same cost as required to meet previous practice. Note, however, that perception can play a significant role in defining what is considered successful past practice. Furthermore, if the data available at the time were limited or not of high enough quality to produce reliable statistics, this would add doubt as to the reliability of the resistance factors determined based on theoretical margins of safety. In many cases, the data available to Barker, et al. was limited in terms of its quantity or quality. For example, much of the footing bearing resistance data were based on small scale model studies. How well these model studies represent reality will naturally put reliability theory results based on such data in a more tentative position when it comes to making the final decision on the magnitude of the resistance factor to recommend. Based on the results provided by Barker, et al., it appears that they considered the level of safety implied by previous design practice to have significant weight in the final resistance factor selection.

## 2.4 Calibration Approach Used by Paikowsky, et al. (2004)

All calibrations conducted by Paikowsky et al. (2004) were focused on the strength limit state. They gathered an extensive database of pile (338 case histories) and shaft load test results (256 case histories) to develop the statistics needed for the reliability analyses they conducted. They assumed that the measurements from full scale field installations adequately addressed all sources of error (i.e., inherent spatial variability, systematic error, and design model error) due to the size of the database, and that full scale measurements were obtained. Hence, there was no need to address systematic error or spatial uncertainty in addition to the error measured from the field measurements as did Barker, et al. (1991).

They used load statistics from Nowak (1999) and the load factors in current use in the AASHTO LRFD specifications (AASHTO 2004). The load statistics used are as shown in Table 4. They assumed that both dead load and live load are lognormally distributed. For the analyses they conducted, they evaluated a range of DL/LL ratios, but settled on a ratio of 2.0 for the final analyses. Note that Nowak (1999) indicates the dead and live load distributions are in fact approximately normal. However, since the COV values for the loads are relatively small and for the resistance relatively large, this difference in the distributions assumed should have little, if any effect on the final results.

**Table 4. Load statistics and factors used by Paikowsky, et al. (2004).**

| Load Type | Bias | Coefficient of Variation | Load Factor Used |
|---|---|---|---|
| Dead load | $\lambda_D = 1.05$ | $COV_{QD} = 0.1$ | $\gamma_{DL} = 1.25$ |
| Live load | $\lambda_L = 1.15$ | $COV_{QL} = 0.2$ | $\gamma_{LL} = 1.75$ |

Paikowsky, et al. (2004) treated the loads and the resistance as the random variables. They evaluated the distribution of the data, comparing the probability density graphs for the actual data to the theoretical normal and lognormal distributions, to choose which distribution function to use to characterize the data. In most cases, they found that the resistance data was lognormally distributed. In general, the database was divided into subsets that were considered to be fully random in nature (how the dataset was subdivided depended upon the specific resistance prediction method used). In addition, data that were more than two standard deviations from the mean value for the subset were in general removed to assess the statistical parameters (i.e., bias, COV, and distribution type). However, in some cases judgment was applied regarding whether or not outlier data was removed.

Paikowsky, et al. (2004) fully relied upon the results obtained from reliability theory to determine resistance factors, though calibration by fitting to ASD was also checked. For reliability theory, they relied upon the results from the advanced reliability methods (i.e., Hasofer and Lind, 1974), which they termed “First Order Reliability Method (FORM).” While they compared the results from FORM to MVFOSM calibration results, they relied on the FORM results.

The target reliability indices used for the Paikowsky, et al. (2004) calibration work were based on a review of historical work on this subject, which included the work by Barker, et al. (1991), among others. They also evaluated, using logic, the number of piles or shafts in a group needed to be considered redundant enough to justify a target reliability index of less than 3.0 to 3.5. They concluded that a target reliability index of 3.0 should be used for shaft and pile groups that contain less than 5 shafts/piles, and that a target reliability index of 2.3 should be used for shaft/pile groups that contain 5 or more shafts/piles. All resistance factors were determined to yield a consistent target β for a specific foundation type and in consideration of whether or not the foundation system is considered to be redundant.

For piles, they grouped the data in the database by soil type, resistance prediction method, and for static analysis methods, by pile type. For piles, for static analyses, the number of load test results in each subgroup upon which the statistics used for reliability analyses were based varied from 4 to 80, though in general, the number of test results in each subgroup was less than 20. For dynamic pile analyses, the number of load test results in each subgroup upon which the statistics used for reliability analyses were based varied from 99 to 384. The minimum number of test results available in a given subgroup needed to be confident in the resistance factor that is obtained from reliability theory was not specifically addressed by Paikowsky, et al. (2004). Since they fully relied upon the reliability theory calibration results, it appears that they assumed enough data were available to be confident in the statistics, with the possible exception of the smallest subgroups.

# 3. Spread Footings

### 3.1 Calibration Results from Previous Studies

The report by Barker, et al. (1991), specifically Appendix A, the source of the current AASHTO LRFD resistance factors for foundation design, is the only source of calibration results for footing geotechnical design considered herein. The overall calibration approach they used is as described in Section 2.3. In general, the data used for footing resistance factor calibration purposes were limited, and were from small scale model tests. The specific data used for calibration purposes are summarized as follows:

1. For footing bearing resistance on sand, statistics were available for the semi-empirical SPT method and the semi-empirical CPT method, both of which were developed by Meyerhof (1956). It appears that the source of the statistical model error data for these two methods was from Meyerhof (1956). For the Rational Theory (Barker, et al., 1991), the source of statistical model error data used was from DeBeer (1970), obtained from small model tests.
2. For footing bearing resistance on clay, for the Rational Theory, the source of the statistical data was from Skempton (1951). No statistics were provided for the CPT method on clay.
3. For sliding resistance, it appears that the primary source of the statistics was from the soil parameter statistics and interface shear strength data. The effect of these statistics on sliding resistance was estimated using the controlling limit state function for sliding.
4. No statistical data were provided for bearing resistance of footings on rock, bearing determined from plate load tests, passive resistance to sliding, and sliding of soil on soil.

Table 5 summarizes the resistance factors developed and provided in NCHRP Report 343 (Barker, et al., 1991). Resistance factors they determined from calibration by fitting to ASD in addition to those they determined using reliability theory are provided, as well as the final values they recommended be used. The values developed using reliability theory are consistent with the target reliability index β of 3.5 as described in Section 2.3. However, based on the analyses they conducted, it should be recognized that the resistance factors determined using calibration by fitting to ASD represent a reliability index that varies from 1.3 to 5.7, depending on the soil type and analysis method used (see Section 2.3). Therefore, the level of safety represented by the resistance factors determined using calibration by fitting may not be consistent with the level of safety represented by the resistance factors determined using reliability theory.

In reviewing the results summarized in Table 5, it appears that the final resistance factors selected were influenced by both the calibration by fitting results and the results from reliability theory, and that some judgment was applied to settle on the final recommendation (see Section 2.3). While the level of safety represented in the final resistance factors they recommended may theoretically not be at the target level (i.e., β of 3.5), they are reasonably close or conservative relative to the target level (theoretically).

### 3.2 Development of Recommended Resistance Factors for Footings

Since no new data are readily available to refine the calibrations done by Barker, et al. (1991) for footings, the recommendations provided by Barker, et al., need only to be updated to current (i.e., LRFD) load factors, and current ASD safety factors for the case of calibration by fitting to ASD. Note that the factors of safety assumed for use in calibration by fitting to ASD by Barker, et al. (1991), as summarized in Table 5, are different than what is prescribed in the current AASHTO Standard Specifications (AASHTO 2002). For footing bearing, only a FS of 3.0 is currently specified in the AASHTO Standard Specifications for all soils, though the AASHTO Standard Specifications do not directly deal with the FS needed for the semi-empirical SPT and CPT methods. For sliding, a FS of 1.5 is currently prescribed in the AASHTO Standard Specifications (2002).

It has been proposed that Rational Theory for bearing capacity as presented in AASHTO (2004) be replaced by conventional bearing capacity theory as presented by Munfakh, et al (2001). While similar to the Rational Theory, the conventional bearing capacity theory allows the combination of cohesive and cohesionless soil properties, allowing c-ϕ soils to be used. The effect this proposed design methodology change has on the reliability based resistance factors determined by Barker, et al. (1991) is unknown.

To develop recommendations for resistance factors that are consistent with the assumptions and recommendations in the current and proposed AASHTO LRFD specifications, the resistance factors have been reanalyzed to reflect the changes identified above, with the exception of the change in bearing resistance design methodology to the method summarized by Munfakh, et al (2001). These results are summarized and compared to the results obtained by Barker, et al. (1991) in Table 5. Reliability theory (Monte Carlo method), using the statistics presented in NCHRP Report 343, was also used by the writer to verify the calibration results obtained by Barker, et al. (see Table 5) as well as to extend those results to the current load factors and load statistics.

For bearing resistance in sand using the semi-empirical SPT and CPT methods, calibration by reliability theory provides resistance factors that are quite close to those that are produced by calibration by fitting to ASD using current safety factors. Therefore, the recommended updated resistance factor of 0.45 as shown in Table 5 provides the theoretical target level of safety without changing the level of safety in current use.

In the specific case of the Rational Theory as applied to footings on sand, it should be noted that NCHRP report 343 did not provide reliability theory calibration results, even though statistical data for this method were provided. The model error data for this case were noted as being from small scale model test results by DeBeer (1970). However, the resistance factors were recommended by Barker, et al. (1991) were different than what one would have obtained using calibration by fitting to ASD. Using reliability theory (Monte Carlo simulation) and the statistics provided in Barker, et al., the resistance factors that result appear to be excessively high. Apparently, the results from calibration by fitting to ASD strongly influenced the resistance factor recommendations for this case in spite of the much higher resistance factors reliability theory produces. Reliability theory results do indicate that the resistance factor needed if CPT data is used to estimate the soil friction angle should be higher than the one needed if SPT data is used to estimate the soil friction angle. In addition, since the calibration by fitting to ASD using current load factors is consistent with the Barker et al. recommended resistance factors for this design method, it is proposed that the resistance factors as proposed by Barker, et al. continue to be used for this method.

For footings on clay, reliability theory results appeared to have minimal influence on the final recommended resistance factor provided by Barker, et al. (1991). However, their reliability theory results indicate that resistance factors higher than those determined through calibration by fitting to ASD should be feasible. Since the reliability theory results indicate that higher resistance factors should be feasible, the final recommended resistance factors proposed in Table 5 for footings on clay are 0.05 higher than the resistance factor determined through calibration by fitting to ASD using current FS values. Therefore, with the exception of the clay CPT empirical method, a resistance factor of 0.50 is recommended. No statistical data was available for the Clay CPT empirical method. Therefore, the 0.45 value determined through calibration by fitting to ASD should be used for that case.

In general for footing design (bearing resistance and sliding), it appears that Barker, et al. (1991) only made minor adjustments in consideration of the reliability theory results relative to the results from calibration by fitting to make the final selection of resistance factors. The writer has taken a similar approach to establish the resistance factors updated to reflect the load factors in use in the current AASHTO LRFD specifications, and the proposed updated design methodology, to be consistent with the approach used by Barker, et al. (1991).

In conclusion, it is recommended that the resistance factors for footings be based on consideration of both the calibration by fitting to ASD approach and the results from reliability theory, as determined using the current load factors, until additional data on footing bearing resistance and sliding can be obtained to verify the accuracy of the statistics used in the reliability theory approach. In all cases, the recommended resistance factors are within 0.05 of the resistance factor that would result from calibration by fitting to ASD. Since no statistical data was provided in NCHRP 343 for plate load testing, footing bearing resistance on rock, passive resistance to sliding, and sliding of soil on soil, resistance factors for these cases were based only on calibration by fitting to ASD, using the current LRFD load factors.

**Table 5. Summary of calibration results for spread footings.**

| Strength Limit State | Soil Test Type | Design Method | ASD FS Used (NCHRP 343) | φ from Calibration by fitting to ASD (NCHRP 343) | φ from Reliability Theory (NCHRP 343) | φ Recom-mended in NCHRP 343 | ASD FS Current Practice (AASHTO 2002) | φ from Calibration by fitting to ASD (Current Practice) | φ from Reliability Theory, Updated to Current γ's | New Recom-mended φ |
|---|---|---|---|---|---|---|---|---|---|---|
| Bearing | SPT | Sand Semi-empirical method (Meyerhof) | 4.0 | 0.37 | 0.49 to 0.53 (0.45[4]) | 0.45 | 3.0 | 0.46 | 0.39[4] | 0.45 |
| Bearing | CPT | Sand Semi-empirical method (Meyerhof) | 2.5 | 0.61 | 0.52 to 0.57 (0.53[4]) | 0.55 | 3.0 | 0.46 | (0.47[4]) | 0.45 |
| Bearing | SPT | Sand Rational theory | *4.0 | 0.37 | (0.70[4]) | 0.45 | 3.0 | 0.46 | 0.61[4] | 0.45[5] |
| Bearing | CPT | Sand Rational theory | *2.5 | 0.61 | (0.96[4]) | 0.50 | 3.0 | 0.46 | 0.88[4] | 0.50[5] |
| Bearing | CPT | Clay Semi-empirical method (Meyerhof) | *2.5 | 0.61 | - | 0.50 | 3.0 | 0.46 | | 0.45 |
| Bearing | CPT | Clay Rational theory | 2.5 | 0.61 | 0.46 to 0.74 (0.63[4]) | 0.50 | 3.0 | 0.46 | 0.56[4] | 0.50[5] |
| Bearing | Lab UU | Clay Rational theory | 2.5 | 0.61 | 0.60 to 0.85 (0.82[4]) | 0.60 | 3.0 | 0.46 | 0.73[4] | 0.50[5] |
| Bearing | Field Vane | Clay Rational theory | 2.5 | 0.61 | 0.60 to 0.97 | 0.60 | 3.0 | 0.46 | | 0.50[5] |
| Bearing | - | Rock, Carter and Kulhawy (1988) | ?[+] | ? | - | 0.60 | 3.0 | 0.46 | | 0.45 |
| Bearing | - | Rock, Kulhawy and Goodman (1987) | ?[+] | ? | - | 0.55 | 3.0 | 0.46 | | 0.45 |
| Sliding | CPT and SPT | Sand, cast-in-place concrete footing | 1.8 | 0.84 | 0.79 | 0.80 | 1.5 | 0.92 | | 0.80 |
| Sliding | CPT and SPT | Sand, precast concrete footing | 1.8 | 0.84 | 0.87 | 0.90 | 1.5 | 0.92 | | 0.90 |
| Sliding | Lab UU | sliding on clay controls | 1.8 | 0.84 | 0.93 to 1.03 | 0.85 | 1.5 | 0.92 | | 0.85 |
| Sliding | Field vane | sliding on clay controls | 1.8 | 0.84 | 0.79 to 0.82 | 0.85 | 1.5 | 0.92 | | 0.85 |
| Sliding | CPT | sliding on clay controls | 1.8 | 0.84 | 0.82 to 0.97 | 0.80 | 1.5 | 0.92 | | 0.85 |
| Sliding | - | Soil and soil | ?[1] | ? | - | 1.0 | 1.5 | 0.92 | | 0.90 |
| Passive resistance | - | All soils | ?[2] | ? | - | 0.50 | 3.0? | 0.46 | | 0.50 |
| Plate load test | - | All soils | ?[3] | ? | - | 0.55 | 2.75? | 0.50 | | 0.55 |

*Based on information obtained from K. Rojiani (personal communication, 2005). [+]Based on the final value selected in NCHRP 343, the implied FS appears to be approximately 2.5 to 2.75. [1]Based on the final value selected in NCHRP 343, the implied FS appears to be approximately 1.5. [2]Based on the final value selected in NCHRP 343, the implied FS appears to be approximately 3. [3]Based on the final value selected in NCHRP 343, the implied FS appears to be approximately 2.75. [4]Comparative or other calibrations conducted by the writer using reliability theory (Monte Carlo Method). [5]The recommended resistance factor is applicable to the method as described by Munfakh, et al (2001).

# 4. Drilled Shafts

## 4.1 Calibration Results from Previous Studies

The report by Barker, et al. (1991), specifically Appendix A, is the source of the current AASHTO LRFD resistance factors for drilled shaft design. The overall calibration approach they used is as described in Section 2.3. Paikowsky, et al. (2004) also performed calibrations and have provided recommendations for resistance factors. The overall calibration approach they used is provided in Section 2.4.

For shafts in clay, the model error statistics used Barker, et al. (1991) were based on shaft load test results reported in Reese and O'Neill (1988), and for shafts in rock, based on load test results reported in Horvath and Kenney (1979). Details regarding the size of the database used to develop these statistics were not reported, but can be found in the source documents. No statistical data was reported for shafts bearing in sand. In general, the shaft data used by Barker, et al. for calibration purposes were somewhat limited. Reese and O'Neill (1988) reported a total of 13 straight-shafted (no bell) drilled shaft case histories in clay and an additional 9 case histories for belled shafts in clay. The remaining 19 case histories reported by Reese and O'Neill (1988) are shafts in layered soils (sand and clay, and in a few cases cemented soils). For shafts socketed into rock, Horvath and Kenney (1979) reported a total of 35 load test case histories. Calibrations using reliability theory were performed by Barker, et al. for the Reese and O'Neill (1988) method (soil), in addition to several other older methods, and for the Horvath and Kenney (1979) and Carter and Kulhawy (1987) methods for shafts in rock.

Table 6 summarizes the resistance factors developed and provided in NCHRP Report 343 (Barker, et al., 1991). The resistance factors they determined from calibration by fitting to ASD in addition to those they determined using reliability theory are provided, as well as the final values they recommended be used. The values developed using reliability theory are consistent with the target reliability index $\beta$ of 2.5 to 3.0 as described in Section 2.3. However, the specific values shown in Table 6 that were derived from reliability theory are for a target $\beta$ of 3.0 (including those resistance factors determined by the writer using the Monte Carlo Method to check the results provided by Barker, et al.), to facilitate comparison to other values in the table. Based on analyses conducted by Barker et al., it should be recognized that the resistance factors determined using calibration by fitting to ASD represent a reliability index that varies from 2.2 to 4.3, depending on the soil type and analysis method used (see Section 2.3). Therefore, the level of safety represented by the resistance factors determined using calibration by fitting may not be consistent with the level of safety represented by the resistance factors determined using reliability theory.

In reviewing the results summarized in Table 6, it appears that the final resistance factors selected by Barker, et al. (1991) were influenced by both the calibration by fitting results and the results from reliability theory (values derived using a target $\beta$ of both 2.5 and 3.0 were considered), and that some judgment was applied to settle on the final recommendation (see Section 2.3). For example, for shafts in clay and for the Horvath and Kenney (1979) method for shafts in rock, the resistance factor for side resistance was increased somewhat since the reliability theory calibration results indicated that a higher resistance factor could be used. However, for the Carter and Kulhawy (1987) method, the resistance factor was reduced somewhat to account for the reliability theory calibration results. In both cases, a value between the calibration by fitting and the reliability theory calibration results was selected. While the level of safety represented in the final resistance factors recommended by Barker, et al. are theoretically not exactly at the target level (i.e., $\beta$ of 2.5 to 3.0), they are reasonably close or conservative relative to the target level (theoretically).

They did not provide resistance factors from calibration by fitting to ASD for shafts bearing in sand. They investigated nominal resistance predictions for several methods for shafts in sand, and obtained widely divergent results. They concluded that they could not confidently recommend a resistance factor based on the information they had at the time.

Paikowsky, et al. (2004) developed a more robust database of load test results for drilled shafts than was available to Barker, et al. (1991). For shafts, Paikowsky, et al. (2004) grouped the data in the database by soil type, resistance prediction method, and the type of construction technique used. The number of shaft load test results in each subgroup upon which the statistics used for reliability analyses were based varied from 9 to 53. Details of the characteristics of each subgroup were not provided, other than general soil type and construction methodology.

Details of the calibration approach used by Paikowsky, et al. are provided in Section 2.4. Calibrations using reliability theory were performed by Paikowsky, et al. for the Reese and O'Neill (1988) method (sand and clay), the O'Neill and Reese (1999) method for shafts in intermediate geo-materials (IGM's), and for the Carter and Kulhawy (1987) method for shafts in rock. IGM's are materials that are transitional between soil and rock in terms of their strength and compressibility, such as residual soils, glacial tills, or very weak rock.

As discussed in Section 2.4, Paikowsky, et al. fully relied on the results of the calibrations conducted using reliability theory. These results are summarized in Table 6. They targeted a reliability index, $\beta$, of 3.0 for shaft groups containing less than five shafts, and 2.3 for shafts in groups of five or more. Since the most common situation for shafts is that they are in smaller groups or are isolated as single shafts, Table 6 provides the resistance factors derived for a $\beta$ of 3.0.

The calibration results obtained by Paikowsky, et al. (2004) for shafts in clay are considerably more conservative than those obtained by Barker, et al. (1991) using reliability theory as updated by the writer using the Monte Carlo Method and current LRFD load factors (i.e., 0.60 versus 0.30), and significantly more conservative than the resistance factor implied by current ASD practice (i.e., 0.55). However, for soil profiles that contain layers of clay and sand, the results obtained by Paikowsky, et al. are much less conservative than their results for shafts in clay (i.e., 0.50 to 0.70). The reasons for this wide divergence in resistance factors for these two subsets of the drilled shaft database were not provided. The calibration results obtained by Paikowsky, et al. (2004) for shafts in rock using the Carter and Kulhawy (1987) method, however, were slightly less conservative than the results obtained by Barker, et al. (1991), but slightly more conservative than the resistance factor implied by current ASD practice (see Table 6).

For larger shaft groups (5 or more shafts), Paikowsky, et al. (2004) determined that resistance factors larger than shown in Table 6, which were derived for a $\beta$ of 3.0, could be used. Targeting a $\beta$ of 2.3 instead of 3.0, they found that the resistance factors for the case of larger shaft groups could be increased in general by approximately 20 percent, to take into consideration the greater redundancy in the foundation system that the greater number of shafts provide.

## 4.2 Development of Recommended Resistance Factors for Shafts

As is true for spread footings, the Barker, et al. (1991) recommended resistance factors for shafts used the LFD load factors available at the time. Since the load factors in the current AASHTO LRFD specifications (AASHTO 2004) have changed relative to the LFD load factors, to provide a common basis for comparison, their resistance factors need to be adjusted to reflect the current load factors. Table 6 provides resistance factors determined by the writer using calibration by fitting to ASD and the current load factors as well as using reliability theory (Monte Carlo method). Reliability theory, using the statistics presented in Barker, et al., were used by the writer to verify their calibration results (see Table 6) as well as to extend those results to the current load factors and load statistics. These updated resistance factors can then be compared to the resistance factors determined by Paikowsky, et al. (2004). However, it must be recognized that the resistance factors determined using calibration by fitting to ASD may not represent the same level of reliability as the Paikowsky, et al. reliability theory based resistance factors. The resistance factors developed using calibration by fitting to ASD do provide a benchmark, however, so that one can see if the resistance factors developed using reliability theory are more or less conservative than previous design practice.

It has been proposed that the Reese and O'Neill (1988) Method contained in the current AASHTO LRFD specifications (AASHTO, 2004) be replaced with an update by O'Neill and Reese (1999). Because the original method was focused on allowable stress design, the service and strength limit states were mixed together in the design of shafts in soil through the use of a base diameter settlement correction factor. The 1999 method better separates these two limit states, and does not include the base diameter settlement correction factor for strength limit state design.

Specific differences between the 1988 and 1999 methods at the strength limit state are as follows:

- For side friction in clay, the 1999 method will produce the same results as the 1988 method up to a shear strength $s_u$ of 150 kPa (3 ksf). At higher shear strengths, on average, the 1999 method will be approximately 10 percent more conservative.
- For end bearing in clay, there is no significant difference between the 1988 and 1999 methods, with the exception of the base diameter settlement correction factor mentioned above. The base diameter settlement correction factor makes the 1999 method less conservative than the 1988 method.
- For side friction in sand, there is no difference between the two methods, with the exception of loose to medium dense sands (i.e., $N_{60} \leq 15$ blows/0.3 m), in which case the 1999 method is more conservative than 1988 method by the ratio $N_{60}/15$, and with the exception of gravels and gravelly sands, in which case the 1999 method is less conservative than the 1988 method by a factor of approximately 1.5 (the specific difference varies with depth).
- For end bearing in sand, the 1999 method is 5 percent more conservative than the 1988 method for $N_{60}$ values of 50 blows/0.3 m or less, and the base diameter settlement correction factor mentioned above is not required at the strength limit state for the 1999 method. The base diameter settlement correction factor makes the 1999 method less conservative than the 1988 method. For denser sands, the soil is treated as an IGM.
- For soils or soft rock considered to be an IGM, there is no comparison between the 1988 and 1999 methods, since the 1988 method did not address IGM's.
- For rock, the 1999 method is significantly different from the 1988 method. A direct conceptual comparison between the two methods is not possible.

Due to these differences between the 1988 and 1999 methods, the design model statistics generated by Barker, et al. (1991) and Paikowsky, et al. (2004) are not directly applicable to the proposed design methodology. The approximate effect these differences will have on the resistance factors needed is dependent on the specific soil conditions present for each case history, and the diameter of each shaft. Information adequate to assess this was not available from NCHRP Report 507. While in general it appears that the 1999 method will produce total shaft resistance values that are within

10 to 20 percent of the 1988 method values, the specific difference cannot be estimated with certainty. For shafts in rock, the recommendations provided by Paikowsky et al. are applicable to the method by Carter and Kulhawy (1987), but not the method for rock as provided by O'Neill and Reese (1999). However, with the exception of shafts in clay, the Paikowsky, et al. results are relatively close to the results obtained by Barker, et al. (1991), and slightly more conservative than the resistance factors determined using calibration by fitting to ASD. Until resistance statistics are developed for the updated O'Neill and Reese (1999) method and the necessary calibrations conducted, it is recommended that the results from calibration by fitting to ASD be used for the 1999 method, with the exception of shafts in clay. Since the differences between the 1988 and 1999 methods are generally minimal for shafts in clay, the calibration results from Paikowsky, et al. must be considered in the selection of a resistance factor for shafts in clay. It is recommended that the resistance factor for shafts in clay be reduced relative to the resistance factor determined from calibration by fitting to ASD to reflect the average of the values recommended for shafts in clay from both studies. The resistance factor recommended for clays, however, should be used with caution, and in consideration of local experience, as these vast differences between the two studies may point to the significant effect that local geology can have on shaft resistance in clay (e.g., ability of the clay to become disturbed due to the construction process, presence of seepage, etc.).

For intermediate geomaterials (IGM's), Paikowsky et al. (2004) did calibrate the O'Neill and Reese (1999) method. Therefore the resistance factors they recommend can be used directly. A resistance factor of 0.60 is recommended for side and 0.55 for base resistance for this case. The slightly lower resistance factor for base resistance in this case is recommended to be consistent with the philosophy that the base resistance should logically be more uncertain than the side resistance as discussed previously.

For the Reese and O'Neill (1988) Method, with the possible exception of shafts in clay, the results from Paikowsky, et al. (2004) can be more heavily relied upon to determine the recommended resistance factors. The results provided by Paikowsky, et al. for shafts in clay need further investigation, considering that their results in clay are so much different than the results they obtained for layered sand and clay, as well as the results for shafts in clay by Barker, et al. (1991). The resistance factors they determined for layered (sand and clay) soils are approximately equal to the resistance factor implied by current ASD practice. Therefore, for the 1988 method, the Paikowsky, et al. resistance factors are in general recommended for shafts in sand (an approximate average between the mixed soil values and the sand values appears appropriate), but a value slightly higher than as recommended by Paikowsky, et al. for shafts in clay should be used, in consideration of the previous results obtained by Barker, et al. (1991). This leads to resistance factors for the Reese and O'Neill (1988) method of 0.45 and 0.40 for side and end bearing in clay, respectively, and 0.55 and 0.45 for side and end bearing in sand, respectively. Note that while these recommendations for the Reese and O'Neill (1988) Method have been developed from a slightly different perspective than the resistance factors developed above for the O'Neill and Reese (1999) method, the magnitudes of the resistance factors for both methods are about the same. Therefore, as indicated in Table 6, the recommended resistance factors are considered applicable to both methods.

For the Carter and Kulhawy (1987) method (shafts in rock), the resistance factor for side resistance should be lowered by 0.05 to better reflect the calibration results. No new calibration results were available for the Horvath and Kenney (1979) method. Based on the statistical data summarized by Barker, et al. (1991), but as updated by the writer to current load factors and load statistics, the resistance factor for that method should be lowered by 0.10, to a value of 0.55. End bearing in rock was not specifically calibrated in either study, though Paikowsky, et al. (2004) included the method as part of the overall determination of shaft resistance in rock. Therefore, the resistance factor for this case will remain at 0.50. For the O'Neill and Reese (1999) method in rock, since this method appears to be quite different from the other methods, it recommended that the resistance factor from calibration by fitting to ASD, 0.55, be used for bearing, and 0.45 for uplift until more detailed calibrations can be performed.

Barker, et al. (1991) differentiated between the magnitude of the resistance factors for side and base resistance due to potential greater uncertainty in the base resistance. To be consistent with that philosophy, it is recommended that the resistance factors for the shaft base be approximately 10 percent more conservative than the resistance factors for the shaft side resistance until more detailed statistical analyses can be conducted.

For uplift, the resistance factors recommended are in general approximately 10 percent less than the resistance factors for side resistance for bearing. Where resistance factors determined using reliability theory were available, those results were considered in making the final resistance factor selection for uplift.

Since no statistical data are available for block failure and uplift of shaft groups, and uplift of belled shafts, resistance factors for these cases should be based on calibration by fitting to ASD, using the current LRFD load factors.

If a load test is conduced to measure the bearing resistance of the drilled shaft, historically, when performing calibration by fitting to ASD, a resistance factor of 0.70 would be used. Recent data on load testing for driven piles has been gathered an analyzed by Paikowsky, et al. (2004). Their analysis, with some limitations, can also be applied to the use of load tests for drilled shafts. Considering that field verification of bearing resistance for production shafts not tested is not possible, a maximum resistance factor of 0.70 is recommended. However, a lower resistance factor determined from Table 9 for sites with variable subsurface conditions should be used. See Section 6.2 for additional details justifying the recommended resistance factors for use of pile load test data for design.

For load tests to evaluate shaft uplift resistance, no data are available to assess an appropriate resistance factor. However, since uplift failure can be abrupt, and since there is no way to field verify the actual resistance available short of conducting a load test, it is recommended that the resistance factor be limited to 0.60 for shaft uplift based on pile load test results. This is also consistent with the philosophy used to establish the shaft uplift resistance factors when using static analysis methods.

These recommendations apply to smaller shaft groups (less than 5 shafts) that inherently lack redundancy. As discussed in Section 4.1, for larger shaft group foundations that have redundancy, the resistance factors recommended in Table 6 may be increased by approximately 20 percent.

**Table 6. Summary of calibration results for drilled shafts.**

| Strength Limit State | Condition and Location | Design Method | ASD FS Used | φ from Calibration by fitting to ASD (NCHRP 343) | φ from Reliability Theory (NCHRP 343) | φ Recom-mended in NCHR P 343 | ASD FS Current Practice | φ from Calibra-tion by fitting to ASD (Current Practice) | φ from Reliability Theory (NCHRP 343 Stats.), Updated to Current γ's | φ from Reliability Theory, Recommended in NCHRP 507 | New Recom-mended φ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Bearing | Side Resistance in clay | α-method (Reese and O'Neill 1988) | 2.5 | 0.61[2] | 0.72 (0.67[4]) | 0.65 | 2.5 | 0.55 | 0.60[4] | 0.24 to 0.28, depending on constr. method for clay (0.30 recommended) | 0.45[5] |
| Bearing | Base Resistance in clay | Total Stress (Reese and O'Neill 1988) | 2.75* | 0.55 | | 0.55 | 2.75* | 0.50 | | 0.24 to 0.28, depending on constr. method (0.30 recommended) | 0.40[5] |
| Bearing | Side Resistance in sand | β-method (Reese and O'Neill 1988) | 2.5 | 0.61[2] | - | - | 2.5 | 0.55 | - | 0.25 to 0.73, depending on constr. method (0.40 recommended) | 0.55[5] |
| Bearing | Base Resistance in sand | Reese and O'Neill 1988 | 2.75* | 0.55 | | - | 2.75* | 0.50 | | 0.25 to 0.73, depending on constr. method (0.40 recommended) | 0.50[5] |
| Bearing | Side and Base Resistance (sand/clay mixed profile) | Reese and O'Neill 1988 | - | - | - | - | 2.5 | 0.55 | - | 0.52 to 0.69, depending on constr. method (0.50 to 0.70 recommended) | 0.55 for side, 0.50 for base[5] |
| Bearing | IGM's | O'Neill and Reese 1999 | | | | | 2.5 | 0.55 | | 0.57 to 0.65, depending on constr. method | 0.60 for side, 0.55 for base |
| Bearing | Side Resistance in rock | Carter and Kulhawy (1988) | 2.5 | 0.61[2] | 0.43 (0.46[4]) | 0.55 | 2.5 | 0.55 | 0.40[4] | 0.45 to 0.49, depending on constr. method | 0.50 |
| Bearing | Side Resistance in rock | Horvath and Kenney (1979) | 2.5 | 0.61[2] | 0.73 (0.63[4]) | 0.65 | 2.5 | 0.55 | 0.55[4] | - | 0.55 |

| Strength Limit State | Condition and Location | Design Method | ASD FS Used | φ from Calibration by fitting to ASD (NCHRP 343) | φ from Reliability Theory (NCHRP 343) | φ Recom-mended in NCHRP 343 | ASD FS Current Practice | φ from Calibra-tion by fitting to ASD (Current Practice) | φ from Reliability Theory (NCHRP 343 Stats.), Updated to Current γ's | φ from Reliability Theory, Recommended in NCHRP 507 | New Recom-mended φ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Bearing | Base Resistance in rock | Canadian Geotechnical Society (1985) | 3.0 | 0.51 | - | 0.50 | 2.5 | 0.55 | - | - | 0.50 |
| Bearing | Base Resistance in rock | Pressure Method (Canadian Geotechnical Society 1985) | 3.0 | 0.51 | - | 0.50 | 2.5 | 0.55 | - | - | 0.50 |
| Bearing | Block failure | Clay | 2.3 | 0.67 | - | 0.65 | 2.5 | 0.55 | - | - | 0.55 |
| Uplift | Side Resistance in clay | α-method (Reese and O'Neill 1988) | 2.5 | 0.61 | - | 0.55[3] | 3.0[3] | 0.46 | - | 0.36 (0.35 recommended) | 0.35[5] |
| Uplift | Belled Resistance in Clay | Reese and O'Neill 1988 | 2.5 | 0.61 | - | 0.50 | 3.0[3] | 0.46 | - | - | 0.45[5] |
| Uplift | Sand | Reese and O'Neill 1988 | 2.5 | 0.61 | - | - | 3.0[3] | 0.46 | - | 0.31 (0.35 recommended) | 0.45[5] |
| Uplift | Sand and Clay | Reese and O'Neill 1988 | | | | | | | | 0.63 (0.35 recommended) | 0.45[5] |
| Uplift | Rock | Carter and Kulhawy (1988) | 2.5 | 0.61 | - | 0.45[3] | 3.0[3] | 0.46 | - | 0.38 (0.35 recommended) | 0.40 |
| Uplift | Rock | Horvath and Kenney (1979) | 2.5 | 0.61 | - | 0.55[3] | 3.0[3] | 0.46 | - | | 0.40 |
| Uplift | Group | All soils | 3.0[3] | 0.61 | - | 0.55[3] | 3.0[3] | 0.46 | - | | 0.45 |
| Bearing | All soils | Load test | 1.9 | 0.80 | - | 0.80 | 2.0 | 0.69 | | See Table 9 | Values in Table 9, but 0.70 or less |
| uplift | All soils | Load test | 1.9 | 0.80 | - | 0.80 | 2.0 | 0.69 | - | | 0.60 |

*Implied in NCHRP 343, due the logic that there is greater uncertainty in the base resistance due to the need for greater deformation to mobilize base resistance.
[2]The value shown in the table is slightly lower than the value shown in Appendix A of Barker, et al. (1991). The difference appears to be the result of using a DL/LL ratio of 2.0 rather than 3.0 when performing the calibration by fitting to ASD.
[3]The resistance factors for uplift are assumed to be 0.1 lower than the resistance factors for skin friction in bearing, as tension unloads soil and decreases shaft diameter.
[4]Comparitive or other calibrations conducted by the writer using reliability theory (Monte Carlo Method).
[5]The recommended resistance factor is applicable to both the Reese and O'Neill (1988) and the O'Neill and Reese (1999) methods.

# 5. Driven Piles – Resistance by Static Analysis

## 5.1 Calibration Results from Previous Studies

The report by Barker, et al. (1991), specifically Appendix A, is the source of the current AASHTO LRFD resistance factors for driven pile foundation design. Only driven pile bearing resistance using static analysis methods was evaluated in that study. The overall calibration approach they used is as described in Section 2.3. Correction factors to account for the type of dynamic analysis performed to verify pile resistance ($\lambda_v$), as provided in the current LRFD specifications (AASHTO 2004) were brought in later from an unknown source, and any calibration that was performed to justify the $\lambda_v$ factors was not available. Paikowsky, et al. (2004) have also performed calibrations and provided recommendations for resistance factors. The overall calibration approach they used is provided in Section 2.4.

Regarding driven piles, Barker, et al. (1991) provided statistics for model error, systematic error in the measurement of the soil parameters, and error due to inherent spatial variability. For additional details regarding data sources used, see Section 2.3. Details regarding the size of the database used to develop these statistics for pile foundations were not reported, but can be found in the source documents. In general, the pile foundation data used by Barker, et al. for calibration purposes were somewhat limited. Calibrations using reliability theory were performed by Barker, et al. for the α-Method (Tomlinson 1987), the λ-Method (Vijayvergiya and Focht 1972), the β-Method (Esrig and Kirby 1979), the SPT Method (Meyerhof 1976) and the CPT Method (Nottingham and Schmertmann 1975).

Table 7 summarizes the resistance factors developed and provided in NCHRP Report 343 (Barker, et al., 1991). Resistance factors determined by Barker, et al. from calibration by fitting to ASD in addition to those they determined using reliability theory are provided, as well as the final values they recommended to be used. The values developed using reliability theory are consistent with the target reliability index β of 2.0 to 2.5 as described in Section 2.3. Table 7 provides the range of resistance factors they obtained from reliability theory for this range of β values. Based on analyses they conducted, the resistance factors determined using calibration by fitting to ASD represent a reliability index that varies from 1.6 to 3.3, depending on the soil type and analysis method used (see Section 2.3). Therefore, the level of safety represented by the resistance factors determined using calibration by fitting may not be consistent with the level of safety represented by the resistance factors determined using reliability theory.

In reviewing the results summarized in Table 7, it appears that the final resistance factors selected were influenced by both the calibration by fitting results and the results from reliability theory (values derived using a target β of both 2.0 and 2.5 were considered), and that some judgment was applied to settle on the final recommendation (see Section 2.3). To make the final selection of resistance factors, Barker, et al. considered the results of previous studies (the specific studies were not mentioned) that indicated the α-Method was most reliable, followed by the λ-Method, and followed by the β-Method as the least reliable of the three static analysis methods. They also considered the adequacy of the database used to obtain the statistics; i.e., for piles in clay, Sidi (1986) reported the results of "numerous" pile load tests. Therefore, for the α-Method, they heavily relied on the reliability theory results, selecting a resistance factor that was higher than the resistance factor implied by past practice (i.e., using calibration by fitting to ASD) and that reflected an approximate lower bound of the reliability theory calibration results for a β of 2.0 and 2.5.

Similarly, for the λ-method, since the reliability theory calibration results indicated that a resistance factor that was slightly lower than the resistance factor implied by past practice should be used, they recommended a resistance factor that was equal to an approximate upper bound of the reliability theory calibration results for a β of 2.0 and 2.5. However, for the β-Method, based on the previous studies mentioned above and experience with the β-Method that indicates different engineers can obtain widely divergent results, they determined, based on their judgment, that a value significantly lower than the reliability theory calibration and the calibration by fitting to ASD results should be used (see Table 7).

Regarding the CPT and SPT in-situ methods for estimating pile bearing resistance, Barker, et al. reasoned that since the CPT will obtain a continuous measurement of the soil profile, where as the SPT obtains a discontinuous profile, the CPT Method should logically be more reliable than the SPT Method. Statistics on the reliability of the measurements obtained using both methods confirmed this assumption. Hence the resistance factor selected for the CPT Method should be higher than the resistance factor for the SPT Method. In general, they relied on the reliability theory calibration results to select recommended resistance factors for these two methods, slightly decreasing the CPT Method resistance factor and slightly increasing the SPT Method resistance factor relative to the resistance factor implied by past practice (i.e., as determined by calibration by fitting to ASD).

Regarding piles bearing in rock, since no statistical data were available, the resistance factor was derived by Barker, et al. (1991) directly from calibration by fitting to ASD. The ASD FS used was the value of 3.0 recommended by the Canadian Geotechnical Society (1985).

For comparison purposes, the resistance factors recommended in the current AASHTO LRFD specifications (AASHTO 2004) are included in Table 7. The AASHTO (2004) resistance factors are the Barker, et al. (1991) recommended values, but multiplied by a $\lambda_v$ factor to account for the field bearing resistance verification method used. The $\lambda_v$ factor for use of a driving formula as the field verification method (i.e., $\lambda_v = 0.80$) has been used to obtain the values shown in Table 7, in the eighth column from the left.

Paikowsky, et al. (2004) developed a more robust database of load test results for driven piles than was available to Barker, et al. (1991). As discussed in Section 2.4, Paikowsky, et al. determined that the load test results adequately accounted for all sources of error. Therefore, there was no need to add in the effects of spatial variability and systematic error as separate sources of error. Paikowsky, et al. (2004) grouped the data in the pile load test database by soil type, resistance prediction method, and pile type. The number of pile load test results in each subgroup upon which the statistics used for reliability analyses were based varied from 4 to 80, but typically were less than 20. Details of the characteristics of each subgroup (with regard to static analysis) were not provided, other than general description of the soil type, pile type, and pile size.

Details of the calibration approach used by Paikowsky, et al. (2004) are provided in Section 2.4. Calibrations using reliability theory were performed by Paikowsky, et al. for the α-Method (Tomlinson 1987), the λ-Method (Vijayvergiya and Focht 1972), the β-Method (Esrig and Kirby 1979), the SPT Method (Meyerhof 1976), the CPT Method (Nottingham and Schmertmann, 1975), and the Nordlund/Thurman Method (Hannigan, et al., 1997), and others.

As discussed in Section 2.4, Paikowsky, et al. (2004) fully relied on the results of the calibrations conducted using reliability theory. The results for pile static analysis methods are summarized in Table 7. They targeted a reliability index, β, of 3.0 for pile groups containing less than five piles, and 2.3 for piles in groups of five or more. Since the most common situation for piles is that they are in relatively large groups, Table 7 provides the resistance factors derived for a β of 2.3.

This value of β is approximately midway between the β range of 2.0 to 2.5 selected by Barker, et al. (1991). The resistance factors from both studies obtained using reliability theory should be approximately comparable with regard to the β values (i.e., margin of safety) they are intended to represent. However, it must also be recognized that the Barker, et al. values were derived using LFD load factors, whereas the Paikowsky et al. values were derived using LRFD load factors (see Table 1). Therefore, to make a more direct comparison of reliability theory derived resistance factors, the Barker, et al. values must be reduced by approximately 10 percent (as has been demonstrated for footings and shafts - see Tables 5 and 6). Furthermore, Paikowsky, et al. subdivided their determination of resistance factors based on pile type, whereas Barker, et al. did not make this distinction. While this complicates the comparison between the two studies, the general trends are still reasonably clear.

Only the resistance factors for what was described as "sand" or "clay" are provided in Table 7. Paikowsky, et al. (2004) also developed resistance factors for piles in mixed soils (i.e., containing layers of clay and sand). Since layered soil profiles require that more than one static analysis method be used, such results do not fit conveniently in a table of resistance factors that are focused on specific methods. This does not mean that such results are not valid, it just means that they add a degree of complexity that the writer has chosen not to specifically address herein. In general, however, the resistance factors for sand and clay layered soils are similar in magnitude to the resistance factors they developed for the sand and clay soil profiles provided in Table 7. For detailed guidance on the selection of resistance factors for piles driven through complex soil stratigraphy, see Paikowsky, et al.

Table 7 shows that the resistance factors obtained by Paikowsky, et al. (2004) are considerably more conservative than the values obtained by Barker, et al. (1991) for all pile bearing methods, with the exception of the Schmertmann CPT method. Both studies yielded similar results for the Schmertmann CPT method. Regarding uplift, it is interesting to note that based on the Paikowsky, et al. results, where a direct comparison between compression and uplift resistance factors for the same method could be made using reliability theory, the uplift resistance factor is approximately 0.05 to 0.10 less than the compression resistance factor. This appears to verify the assumption made by Barker, et al. to establish uplift resistance factors relative to the calibrated compression loading resistance factor values.

The resistance factors provided in Table 7 and the observations made herein apply to larger pile groups (5 or more) where redundancy in the pile foundation can be relied upon. For smaller pile groups (less than 5 piles), Paikowsky, et al. (2004) determined that resistance factors smaller than shown in Table 7, which were derived for a β of 2.3, should be used. Targeting a β of 3.0 instead of 2.3, they found that the resistance factors for the case of smaller pile groups should be decreased in general by approximately 20 percent, to take into consideration the lack of redundancy in the foundation system.

### 5.2 Development of Recommended Resistance Factors for Piles – Static Analysis Methods

As discussed in Section 5.1, the resistance factors developed by Barker, et al. (1991) using calibration by fitting to ASD and by reliability theory, as well as the resistance factors in the current AASHTO LRFD specifications (AASHTO 2004), were derived using LFD load factors rather than LRFD load factors. However, the Paikowsky, et al. (2004) resistance factors were developed using the current LRFD load factors. Updated resistance factors determined using calibration by fitting to ASD and LRFD load factors are provided in Table 7. The safety factor used in this case as the basis for calibration by fitting to ASD, current practice, is 3.0 (with the exception of the Meyerhof SPT method, which historically has used 4.0), as discussed in Hannigan, et al. (1997). This safety factor is described by Hannigan. et al. as what has been used historically, though they recommend that the final safety factor used for pile foundation design directly consider the method used to field verify the pile resistance obtained (see Section 6.1 for additional discussion on this point). The resistance factors from calibration by fitting to ASD may not represent the same level of reliability as the Paikowsky, et al. reliability theory based resistance factors. The resistance factors developed using calibration by fitting to ASD do provide a benchmark, however, so that one can see if the resistance factors developed using reliability theory are more or less conservative than previous design practice.

Paikowsky, et al. (2004) have recommended that the resistance factors for static analysis of piles differentiate between pile types (precast concrete, steel pipe, and H-piles). While in some cases, there are clear differences in the magnitude of the resistance factors they obtained using reliability theory, a clear pattern in the results that would give some indication of the reason for the differences was not present. In general, it appears that, based on their report, different soil shear strength correlations and soil property averaging techniques were used for different pile types designed using the same design method. This creates difficulties in making direct comparisons between pile types to assess how much difference in the resistance factors for different pile types should exist. Paikowsky, et al. indicate that subdividing the database by both pile and soil type resulted in data subsets that in some cases were really too small to be reliable statistically. This further complicates the problem of interpreting the differences in resistance factor magnitudes for different pile types. Because of these issues, recommended resistance factors provided in Table 7 do not specifically differentiate between pile types. However, regarding the design methodologies investigated, the α-Tomlinson Method (clays), the β-Method (clays), and the λ-Method (clays) do not differentiate between pile types for their assessment of pile skin friction. Intuitively, there should be at least a minor difference between the various pile types regarding the skin friction. Therefore, it is recommended that designers be made aware of the more detailed data and resistance factor recommendations in NCHRP Report 507, to assess on a project specific basis whether or not the selected resistance factor should be differentiated based on pile type.

The reason for the rather large difference between the two studies regarding the resistance factor magnitudes pointed out in Section 5.1 is unknown. In addition, the resistance factors determined by Paikowsky, et al. (2004) using reliability theory are also significantly less than the resistance factor implied by past ASD practice (i.e., calibration by fitting to ASD), especially for piles in clay. Regarding the β-Method, Barker, et al. (1991) recognized that at least for that method, their calculated resistance factor based on reliability theory seemed high in light of previous experience with that method. Therefore, to some extent, the low resistance factors obtained by Paikowsky, et al. for the β-method are not a complete surprise. For piles in sand, using the Nordlund/Thurman Method, and for the Meyerhof SPT and Schmertmann CPT methods, the resistance factors obtained by Paikowsky, et al. are reasonably close to the resistance factor implied by previous ASD practice.

Recommended resistance factors are provided in Table 7 (last column of the table). The resistance factors for piles in clay have been reduced relative to the values obtained by Barker, et al. (1991) as well as those implied by current ASD practice, due to the more conservative results obtained by Paikowsky, et al. (2004) for those cases. For the other methods, minor adjustments in the resistance factors relative to the values implied by past ASD practice have been made to reflect the calibration results by Paikowsky, et al. For uplift resistance, the Paikowsky, et al. results were used to establish resistance factor values for those methods where calibration results were available. For the other uplift estimation methods, the recommended resistance factors for piles in compression were reduced by 0.10 to be consistent with previous derivation of uplift resistance factors by Barker, et al. (1991).

These recommendations apply to pile groups of 5 or more that inherently possess some degree of redundancy. As discussed in Section 5.1, for smaller pile group foundations that lack this redundancy, the resistance factors recommended in Table 7 should be reduced by approximately 20 percent.

### 5.3 Resistance Factor Application Issues for Static Analysis of Pile Resistance

Note that the resistance factors for static pile bearing resistance should only be used for final sizing of the pile foundation for bearing if the pile resistance is not field verified using a dynamic method. In this scenario, the pile resistance is determined by the static analysis method at a specific tip elevation, and is then driven to that specific tip elevation without using a driving formula or dynamic measurements such as from a pile driving analyzer combined with signal matching – in this case the reliability of the pile resistance is dependent on the reliability of the static analysis method used.

The current LRFD specifications imply that the reliability of the nominal pile resistance is a combination of the reliability of the static analysis method used and the field resistance verification method used through the use of the $\lambda_v$ factor. However, if the pile resistance is field verified using a dynamic method, the reliability of the pile resistance is fully dependent on the reliability of the dynamic method used to verify the pile resistance, and the resistance factor developed for the dynamic method and its associated resistance factor should be used to determine the number of piles of a specified feasible nominal resistance that is required to support the applied factored loads for the various limit states. The static analysis method, in that case, is only used to establish a feasible resistance as a starting point for the pile foundation design and to estimate pile length for contract quantities. The actual location where the pile tip is stopped is dependent only on the field verification of pile bearing resistance, provided the pile tip is below any established minimum tip elevation requirement resulting from settlement, lateral resistance, uplift, scour, or downdrag considerations.

**Table 7. Summary of calibration results for driven piles, static analyses.**

| Condi-tion and Location | Design Method | Pile Type | ASD FS Used NCHRP 343 | φ from Calibration by fitting to ASD (NCHRP 343) | φ from Reliability Theory (NCHRP 343) | φ Recom-mended in NCHRP 343 | φ Reduced by $\lambda_v$-Factor for Driving Formula in AASHTO 2004 | ASD FS Current Practice | φ from Calibration by fitting to ASD (Current Practice) | φ from Reliability Theory, NCHRP 507 | New Recom-mended φ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Bearing in clay | α-Tomlinson method | concrete | 2.5 | 0.61 | 0.70 to 0.90[1] | 0.70 | 0.56 | 3.0 | 0.46 | 0.36 | 0.35 |
| | | pipe | | | | | | | | 0.25 | |
| | | H-Pile | | | | | | | | 0.40 | |
| Bearing in clay | λ-method | concrete | 2.5 | 0.61 | 0.49 to 0.62[1] | 0.55 | 0.44 | 3.0 | 0.46 | 0.48 | 0.40 |
| | | pipe | | | | | | | | 0.24 | |
| | | H-Pile | | | | | | | | 0.37 | |
| Bearing, in clay | β-method | concrete | 2.5 | 0.61 | 0.68 | 0.50 | 0.40 | 3.0 | 0.46 | 0.32 | 0.25 |
| | | pipe | | | | | | | | 0.14 | |
| | | H-Pile | | | | | | | | 0.19 | |
| Bearing in sand | Nordlund/ Thurman | concrete | - | - | - | - | - | 3.0 | 0.46 | 0.42 | 0.45 |
| | | pipe | | | | | | | | 0.56 | |
| | | H-Pile | | | | | | | | 0.46 | |
| Bearing in sand | Meyerhof SPT | concrete | 4.0 | 0.38 | 0.46 to 0.49[1] | 0.45 | 0.36 | 4.0 | 0.34 | 0.19 | 0.30 |
| | | pipe | | | | | | | | 0.31 | |
| | | H-Pile | | | | | | | | 0.42 | |
| Bearing, all soils | Schmertmann CPT | All piles | 2.5 | 0.61 | 0.54 to 0.57[1] | 0.55 | 0.44 | 3.0 | 0.46 | 0.51 | 0.50 |
| Bearing in rock | Canadian Geo. Society, 1985 | All piles | 3.0 | | - | 0.50 | 0.40 | 3.0 | 0.46 | - | 0.45 |
| Uplift | α-Tomlinson-method | All piles | - | 0.51* | - | 0.60 | 0.60 | - | 0.36* | 0.23 - 0.33 | 0.25 |
| Uplift | λ-method | All piles | - | 0.51* | | 0.45 | 0.45 | - | 0.36* | 0.27 | 0.30 |
| Uplift | β-method | All piles | - | 0.51* | - | 0.40 | 0.40 | - | 0.36* | 0.12 – 0.19 | 0.20 |
| Uplift | Nordlund-method | All piles | - | - | - | - | - | - | 0.36* | | 0.35* |
| Uplift | Meyerhof SPT | All piles | - | 0.30* | | 0.35 | 0.35 | - | 0.24* | - | 0.25 |
| Uplift | Schmertmann CPT | All piles | - | 0.51* | | 0.45 | 0.45 | - | 0.36* | - | 0.40* |

*Estimated by reducing the resistance factor for compression loading by 0.10.
[1]K. Rojiani (personal communication, Feb. 2005).

# 6. Driven Piles – Resistance by Dynamic Analysis, and Other Pile Design Resistance Factors

### 6.1 Calibration Results from Previous Studies

As discussed in Section 5.3, pile bearing resistance uncertainty is the direct result of the uncertainty in the method used during pile driving to verify the bearing resistance. For example, if a reasonably accurate static analysis method is used to estimate the pile resistance, such as the Nordlund/Thurman Method or the Schmertmann CPT method, but a relatively inaccurate dynamic method such as the ENR driving formula (discussed later in this section) is used the verify pile bearing in the field and is used as the basis for determining the final tip elevation for the pile, it is obvious that the uncertainty in the ENR formula must be considered when determining a resistance factor to use to size the pile foundation. This becomes especially obvious if the pile tip is stopped, for example, 6 m (20 ft) above the tip elevation determined from the static analysis method to obtain the desired bearing resistance. This approach is consistent with previous ASD pile design practice as described in AASHTO (2002), in that these specifications specifically state "the selection of the factor of safety to be applied to the ultimate axial geotechnical capacity shall consider the reliability of the ultimate soil capacity determination and pile installation control." The only pile design safety factors provided in AASHTO (2002) include some form of pile resistance field verification. The largest safety factor specified is for the case where a dynamic formula is used, and the safety factors specified decrease as the reliability of the field resistance verification method increases.

Calibration of resistance factors for pile bearing resistance using dynamic methods (e.g., driving formula, wave equation, pile driving analyzer with signal matching) was not conducted as part of NCHRP 343 by Barker, et al. (1991). They did, however, recommend based on judgment to use a resistance factor for the pile driving analyzer with signal matching of 0.70, and for pile bearing as determined from a load test, based on calibration by fitting to a ASD safety factor of 1.9, a resistance factor of 0.80. These resistance factors were recommended assuming that LFD load factors are used, which, as discussed previously, are not consistent with the current LRFD load factors.

Paikowsky, et al. (2004) did perform reliability theory calibrations and provided recommendations for resistance factors for pile bearing resistance using dynamic methods or load testing. The overall calibration approach they used is provided in Section 2.4. Paikowsky, et al. used the load test results discussed in Section 5.1 to develop statistics for these calibrations. For dynamic pile analyses, the number of load test results in each subgroup upon which the statistics used for reliability analyses were based varied from 99 to 384. The specific subgroup size depended on the method being analyzed (i.e., not all case histories had adequate data to perform calibrations on all the dynamic analyses), and whether or not end of driving (EOD) data, beginning of redrive (BOR) data, or data from both sources were used. EOD and BOR data were in general combined into one dataset to obtain statistics for the driving formulae, whereas for the other dynamic methods these two types of data were treated separately. The main difference between EOD and BOR data is in the driving resistance measured or the dynamic measurements taken, which are key input values to estimate bearing resistance for these methods. The BOR data allows one to assess, at least approximately, how much soil setup or relaxation occurred after initial driving. Note also that the EOD and BOR data were usually taken from the same pile, but at different times, and both measurements apply to one load test result. Therefore, the EOD and BOR data are not really statistically independent values, since they are both related to the same load test for a given pile tested. Also note that some further subdivision of these subgroups was done by Paikowsky, et al. in some cases to evaluate the effect of the magnitude of driving resistance and pile volume. In those cases, as few as 37 load test results were evaluated to obtain the statistics.

As discussed in Section 2.4, Paikowsky, et al. (2004) fully relied on the results of the calibrations conducted using reliability theory. The results for pile dynamic analysis methods are summarized in Table 8. They targeted a reliability index, β, of 3.0 for pile groups containing less than five piles, and 2.3 for piles in groups of five or more. Since the most common situation for piles is that they are in relatively large groups, Table 8 provides the resistance factors derived for a β of 2.3.

Pile load test data were used by Paikowsky. et al. (2004) as the baseline of comparison to develop the statistics needed to perform reliability analysis to determine resistance factors. The reliability of the pile load test data to characterize the pile resistance at a site depends on the number of pile load tests conducted at the site and the variability of the conditions within the site. The greater the number of tests conducted, the more reliable is the measured nominal resistance. Paikowsky, et al. attempted to quantify site variability and then tie that variability to the number of load tests needed to justify a specific resistance factor for foundation design (see Table 9). For this application, a site is defined as an area where the subsurface conditions can be characterized as geologically similar, both in terms of subsurface stratification and the engineering properties of the strata. See Paikoswky, et al. (2004) for details regarding a suggested approach to characterizing the site variability.

They did a similar analysis to determine the number of dynamic measurements (e.g., Pile Driving Analyzer, or PDA) with a signal matching analyses (e.g., CAPWAP) that are needed per site, in consideration of the site variability, to justify the resistance factor they recommended for that methodology. The results of that analysis are provided in Table 10.

The calibrations for dynamic measurements and load tests done by Paikowsky et al. (2004) assume that the pile hammer used for the load test(s) is the same as the hammer used for the production pile driving. In this case, pile acceptance is based on driving the pile to at least the depth of the pile tip in the load test, possibly adjusting that tip elevation across the substructure unit(s) based on known stratigraphy from boring data, and driving the pile to the minimum blow count and hammer stroke observed at bearing for the pile load test (S. Paikowsky, personal communication 2005). Therefore, the calibrations they conducted did not take into account the potential to use the wave equation and/or PDA calibrated to the load test and/or PDA with signal matching results as a means to apply the those results to the production piles not specifically tested.

The resistance factors provided in Table 8 apply to larger pile groups (5 or more) where redundancy in the pile foundation can be relied upon. For smaller pile groups (less than 5 piles), Paikowsky, et al. (2004) determined that resistance factors smaller than shown in Table 7, which were derived for a β of 2.3, should be used. Targeting a β of 3.0 instead of 2.3, they found that the resistance factors for the case of smaller pile groups should be decreased in general by approximately 20 percent, to take into consideration the lack of redundancy in the foundation system.

### 6.2 Development of Recommended Resistance Factors for Piles – Dynamic Analysis Methods and Group Resistance

Resistance factors determined using calibration by fitting to ASD and LRFD load factors conducted by the writer are provided in Table 8. The safety factors used as the basis for calibration by fitting to ASD are provided in AASHTO (2002), and varied from 1.9 to 3.5, depending on which method is selected to verify pile resistance during or after pile driving (see Section 6.1 for additional discussion on this point). The resistance factors from calibration by fitting to ASD may not represent the same level of reliability as the Paikowsky, et al. (2004) reliability theory based resistance factors. The resistance factors developed using calibration by fitting to ASD do provide a benchmark, however, so that one can see if the resistance factors developed using reliability theory are more or less conservative than previous design practice.

In addition to the calibration results provided by Paikowsky, et al. (2004), reliability analyses using Monte Carlo simulations were conducted by the writer, using the same database of pile load test results as was used by them. Both reliability analysis approaches provided similar results in most cases, with the exception of the reliability analyses for the Engineering News Record (ENR) formula. Of the dynamic methods calibrated by Paikowsky, et al., and by the writer using the Monte Carlo Method, the minor differences that occurred are likely the result of differences in how the lognormal distributions were fit to the actual data.

The Engineering News Record formula was originally developed as an allowable stress design method, and contained within the formula a factor of safety of 6 (Peck, et al., 1974). The ENR equation as reported by Peck, et al. is specifically as follows:

$$R_a = \frac{W_H H}{FS(s+C)} \tag{8}$$

where,
$R_a$ = allowable (working) pile resistance measured during driving
$W_H$ = weight of the hammer ram, expressed in the same units as $R_a$
H = height of fall of the ram (i.e., its stroke), expressed in the same units as s and C
s = pile permanent set, (mm or IN)
C = energy loss per hammer blow, (2.5 mm or 0.1 IN for all hammers except drop hammers, and 25 mm or 1.0 IN for drop hammers)
FS = factor of safety, recommended as 6.0.

Note that $W_H H = E_d$ = developed hammer energy. This is the kinetic energy in the ram at impact for a given blow. If ram velocity is not measured, it may be assumed equal to the potential energy of the ram at the height of the stroke, taken as the ram weight times the stroke.

To perform a true calibration for LRFD, built in safety factors must be removed so that a nominal resistance is calculated. Therefore, if the safety factor is removed,

$$R_{ndr} = \frac{W_H H}{(s+C)} \tag{9}$$

where, $R_{ndr}$ = nominal pile resistance measured during driving.

Paikowsky, et al. (2004) performed the calibration using ENR with the factor of safety of 6 included, causing the resistance factor they determined to be higher than what would be obtained if the FS of 6 is removed. The calibration performed by the writer used a bias that is obtained when the factor of safety of 6 is removed from the formula, and using the EOD data provided by Paikowsky, et al., results in a resistance factor of 0.10. While this appears to be substantially lower than past practice, in reality it is not, since the built in factor of safety of 6 was removed from the equation for this calibration. However, note that the poor correlation of the ENR formula pile bearing resistance has long been recognized (Peck, et al., 1974). Peck, et al. recommended that further use of the formula is not justified because of this. Therefore, the low resistance factor is not a surprise.

The only other driving formula calibrated by Paikowsky, et al. (2004) is the FHWA Gates formula (Hannigan, et al., 1997). The resistance factor they obtained is consistent with the level of safety that would be obtained from past practice, and is therefore preferred over the ENR formula, if a driving formula is used. The resistance factor provided in Table 8 for this method was developed using only EOD data, though Paikowsky, et al. also performed a calibration that included the EOD and BOR data together in one data set. They found that, regarding the inclusion of BOR data, the differences relative to only including the EOD data were relatively minor in this case.

With the exception of the resistance factors for the wave equation and the ENR formula, the resistance factors for the dynamic pile methods determined by Paikowsky, et al., are close to the resistance factors implied by past ASD practice (i.e., calibration by fitting to ASD). Therefore, the proposed resistance factors should not significantly change the degree of conservatism in pile foundation design for those methods.

The resistance factor for the wave equation analysis provided in Table 8, for the case where load test data or dynamic test data is not available, is somewhat lower than what has been used in past practice. Note that without dynamic test results with signal matching analysis and/or pile load test data, considerable judgment is required to use the wave equation to predict the pile bearing resistance. Key soil input values that affect the predicted resistance include the soil damping and quake values, the skin friction distribution, and the anticipated amount of soil setup or relaxation. Furthermore, the actual hammer performance is a variable that can only be accurately assessed through dynamic measurements, though "standard" input values are available. The Paikowsky, et al. (2004) wave equation calibrations were performed by using the wave equation with standard input values. Therefore, their wave equation calibrations did not consider the potential improved pile resistance prediction reliability that could result from measurement of at least some of these key input values.

To establish resistance factors for dynamic measurements with signal matching and pile load tests, how the data are used for pile resistance verification and for development of the final driving criteria must be considered. For example, some type of pile resistance verification method must be used in conjunction with the pile load test results to extrapolate those results to production piles not load tested. In the AASHTO Standard Specifications (2002), representing ASD practice, the pile load test results are either combined with a wave equation analysis to produce driving criteria for the production piles not load tested, or the pile load test results are combined with wave equation analyses and dynamic measurements with or without signal matching to produce driving criteria for the production piles not load tested. If a load test is not conducted, but dynamic measurements with signal matching analyses are performed instead, a similar process to extrapolate the test pile measurements to the production piles not tested is used. In this case, AASHTO (2002) recommends that the wave equation be used to extrapolate the test pile results to the production piles not tested. In both cases, extrapolation is accomplished through calibration of the wave equation and/or signal matching analyses to yield a resistance that is consistent with what was obtained from the test pile(s).

With regard to the combination of pile load test results and dynamic tests with signal matching analyses, the reliability of the pile nominal resistance cannot be improved beyond the reliability of the pile load test(s) themselves. Even though the signal matching analysis can be calibrated to match the pile load test measured resistance, there is the potential for some uncertainty to be introduced into the pile resistance prediction relative to simply conducting more pile load tests. This is especially true if the hammer used to install the load test pile is different, resulting in large differences in impact velocity, than the hammer used for the production piles (e.g., use of a steam hammer versus a single acting diesel hammer). Hence, conducting dynamic tests with calibrated signal matching analyses cannot be used to increase the reliability of the pile foundation, but can be used to maintain the reliability of the foundation resistance determination for all of the production piles based on the pile load test results.

A good quality pile inspection program and judicious selection of production piles to be dynamically monitored can go a long way to keeping the extrapolation uncertainty (i.e., extrapolation to production piles not load tested or dynamically tested) to a minimum. If dynamic testing of production piles is not conducted, application of the load test, or dynamic test with signal matching results, to the remaining production piles is fraught with assumptions that can potentially add additional variability not considered in the calibrations conducted by Paikowsky, et al. For example, even if the hammer stroke is the same, based on the writer's experience and the experience of others (e.g., Hannigan, et al., 1997), there can be wide variations in transferred energy that will affect the pile resistance obtained at a given blow count. Dynamic testing of production piles provides a direct verification of the energy transferred to the pile, improving the accuracy of the extrapolation of the driving criteria developed from the test pile results to the production piles. Intuitively, using wave equation analysis combined with dynamic testing to verify the transferred energy from the hammer to the production pile should improve the reliability of the production pile criteria developed from the test pile relative to criteria developed from the test pile without this additional verification data. However, specific calibration results are not available for the case where dynamic testing combined with wave equation analysis is used to extrapolate test pile results to production piles. Therefore, it is recommended that the resistance factors provided in tables 9 and 10 be applied to both scenarios until additional calibrations are conducted.

The recommended resistance factors provided in Table 8 (last column of the table) fully rely upon the reliability theory calibration results. For uplift load tests, no data are available to assess an appropriate resistance factor. However, since uplift failure can be abrupt, and since there is no way to field verify the actual resistance available short of conducting a load test, it is recommended that the resistance factor be limited to 0.60 for pile uplift based on pile load test results. This is also consistent with the philosophy used to establish the pile and shaft uplift resistance factors when using static analysis methods (see Sections 4.2 and 5.2).

These recommendations apply to pile groups of 5 or more that inherently possess some degree of redundancy. As discussed in Section 6.1, for smaller pile group foundations that lack this redundancy, the resistance factors recommended in Table 8 should be reduced by approximately 20 percent.

Resistance factors are also provided in Table 8 for block failure of pile groups in clay and pile group uplift. A relatively low FS was used by Barker, et al. (1991) for calibration by fitting to ASD to obtain the resistance factor for this limit state, because this mode of failure rarely controls. Using LFD load factors, the resistance factor obtained by calibration by fitting to ASD was recommended in that report to be 0.65. The recommended resistance factor of 0.60 for this case as provided in the table is based on current LRFD load factors rather than LFD load factors. For group uplift, neither study provided a recommendation. Therefore, lacking any statistical data, calibration by fitting was used by the writer to obtain a recommended resistance factor for this case.

**Table 8. Summary of calibration results for bearing of driven piles, dynamic analyses, and other strength limit states.**

| Limit State | Resistance Determination Method/Condition | ASD FS Current Practice | φ from Calibration by fitting to ASD (Current Practice) | φ from Reliability Theory, NCHRP 507 | φ from Reliability Theory, Monte Carlo[2] | New Recommended φ |
|---|---|---|---|---|---|---|
| Bearing | Driving criteria established by static load test(s), production pile quality control by calibrated wave equation, or minimum driving resistance combined with minimum delivered hammer energy from the load test(s). For the last case, the hammer used for the test pile(s) shall be used for the production piles. | 2.0 | 0.69 | See Table 9 | - | Values in Table 9 (varies from 0.55 to 0.90) |
| Bearing | Driving criteria established by static load test(s), production pile quality control by dynamic testing of at least one production pile per pier at BOR, but no less than the number of tests per site provided in Table 10. Quality control of remaining piles by calibrated wave equation. | 1.9 | 0.72 | - | - | Values in Table 9 (varies from 0.55 to 0.90) |
| Bearing | Driving criteria established by dynamic test with signal matching at BOR, of at least one production pile per pier, but no less than the number of tests per site provided in Table 10. Production pile quality control of remaining piles by calibrated wave equation. | 2.25 | 0.61 | 0.65 | 0.71 | 0.65 |
| Bearing | Wave equation analysis at EOD without pile dynamic measurements or load test | 2.75 | 0.50 | 0.39 | - | 0.40 |
| Bearing | FHWA-Modified Gates pile formula at EOD | 3.5 | 0.39 | 0.38 | 0.44[2] | 0.40 |
| Bearing | Engineering News Record dynamic pile formula at EOD, with built in FS of 6 removed from formula (see Equation 9) | 3.5 | 0.39[3] | 0.26[3] | 0.10[2] | 0.10 |
| Block Bearing Failure | Clay only | 2.3[+] | 0.60 | - | - | 0.60 |
| Uplift | Pile uplift load test | 2.0 | 0.69 | - | - | 0.60 |
| Group Uplift Resistance | All soils | - | 0.50 | - | - | 0.50 |

[+]As recommended in NCHRP Report 343.
[2]Comparitive or other calibrations conducted by the writer using reliability theory (Monte Carlo Method), using statistical data from Paikowsky, et al. (2004) and a target β of 2.3.
[3]FS of 6 was not removed from formula (i.e., formula left as an allowable stress formula, as shown in Equation 8).

**Table 9. Relationship between Number of Load Tests Conducted per Site and φ (after Paikowsky, et al., 2004)**

| Number of Load Tests per Site | Resistance Factor, φ | | |
|---|---|---|---|
| | Site Variability* | | |
| | Low* | Medium* | High* |
| 1 | 0.80 | 0.70 | 0.55 |
| 2 | 0.90 | 0.75 | 0.65 |
| 3 | 0.90 | 0.85 | 0.75 |
| $\geq$ 4 | 0.90 | 0.90 | 0.80 |

*See Paikowsky, et al. (2004) for guidelines on how to assess site variability.

**Table 10. Number of Dynamic Tests with Signal Matching Analysis to Be Conducted During Production Pile Driving (after Paikowsky, et al., 2004).**

| Site Variability* | Low* | Medium* | High* |
|---|---|---|---|
| Number of Piles Located within Site | Number of Piles with Dynamic Tests and Signal Matching Analysis Required (BOR) | | |
| $\leq$ 15 | 3 | 4 | 6 |
| 16-25 | 3 | 5 | 8 |
| 26-50 | 4 | 6 | 9 |
| 51-100 | 4 | 7 | 10 |
| 101-500 | 4 | 7 | 12 |
| > 500 | 4 | 7 | 12 |

*See Paikowsky, et al. (2004) for guidelines on how to assess site variability.

# 7. Determination of Load Factors for Downdrag on Deep Foundations

Downdrag loads can add additional uncertainty when calculating the magnitude of pile resistance needed to resist all the applied loads and achieve the desired level of reliability. The calibration results reported in the previous section assume that only two loads are present, dead load and live load. To date, calibrations have not been conducted using reliability theory for the load case that includes downdrag loads.

In current ASD practice, a nominal value of downdrag load is applied, with the overall safety factor for design of the deep foundation remaining the same as would be used if downdrag loads are not present. In the current AASHTO LRFD specifications (AASHTO 2004), a load factor of 1.8 is specified to be applied to the nominal downdrag load estimate.

Reliability theory using the Monte Carlo method was used by the writer to investigate the magnitude of the load factor needed to account for the additional uncertainty introduced in the deep foundation design due to the downdrag load estimate. Statistics (bias and COV) used were taken directly from Paikowsky, et al. (2004) for the specific cases investigated. The statistics used are summarized in Table 11. Dead load and live load are considered to be normally distributed. The downdrag load and the resistance were considered to be lognormally distributed. A dead load to live load ratio of 3.0 was assumed for all simulations. The downdrag load to dead load ratio was varied to investigate the effect this ratio has on the load factor required.

For piles, two general cases were investigated to estimate the load factor that would be needed: a concrete pile bearing in sand, with the soil producing the downdrag consisting of clay, and a steel pipe pile bearing in sand, with the soil producing the downdrag consisting of clay. For shafts, two typical cases were considered: (1) a shaft bearing in sand, with the soil producing the downdrag consisting of clay, and the shaft is constructed using mixed means (e.g., a combination of casing, slurry, and/or open hole), (2) a shaft bearing in mixed soils (sand and clay), with the soil producing the downdrag consisting of clay, and the shaft is constructed using mixed means (e.g., a combination of casing, slurry, and/or open hole).

The nominal pile resistance available to support structure loads plus downdrag is estimated by considering only the positive skin and tip resistance below the lowest layer contributing to the downdrag. In general, the available factored geotechnical resistance should be greater than the factored loads applied to the pile, including the downdrag, at the strength limit state. This downdrag load is typically estimated as the skin friction in the downdrag zone, determined using a static analysis method such as the $\alpha$-Tomlinson Method or the $\lambda$-Method. Uplift resistance statistics were available for the $\alpha$-Tomlinson Method and the $\lambda$-Method for steel pipe piles, but only total pile compression resistance statistics (i.e., skin friction plus end bearing) were available for concrete piles. While it would be desirable to separate out the end bearing component from the compression resistance statistics, such data was simply not available. Fortunately for most pile in clay situations, the percentage of the load carried in end bearing is typically low. Therefore, it was felt that the compression resistance statistics were usable for this calibration.

For shafts, the bearing resistance is also estimated using static analyses. As stated in the section on shaft design calibration, the shaft strength limit state compression resistance calibration conducted by Paikowsky, et al. (2004) included a settlement correction that was part of the Reese and O'Neill (1988) method (note that the original shaft design method was developed in consideration of ASD methodology). The statistics for shaft bearing resistance provided by Paikowsky, et al. (2004) include this settlement correction. The effect this settlement correction had on the shaft bearing resistance statistics is not known. Paikowsky, et al. (2004) also provided statistics for shaft skin friction without end bearing. This skin friction only data could be used directly to assess the uncertainty in the skin friction calculation method, and therefore could also be used for estimating the uncertainty in the downdrag load calculation.

For both piles and shafts, the uncertainty in the downdrag load was assumed to be represented by the bias and COV of the method used to estimate the skin friction contributing to downdrag. Two magnitudes of downdrag were investigated, as it was found that the magnitude of the downdrag relative to the other loads had a significant influence on the downdrag load factor required. Typically, downdrag loads are on the order of 50% of the applied dead load. Based on the writer's experience, downdrag loads equal to the dead load in magnitude can occur. However, it is usually not practical to design for downdrag loads much greater than this.

The calibration approach used was to determine the resistance factor needed to obtain a target β of 2.3 for piles and 3.0 for shafts if no downdrag load is present, and then hold that resistance factor and target β constant in subsequent Monte Carlo simulations in which downdrag load was present, increasing the load factor for downdrag as needed to achieve the target β. The results are provided in Table 11, and these results should be considered representative of what is typically required to achieve the target β without having to make adjustments to the resistance factors derived for the case where downdrag is not present. Using the resistance factor needed to achieve the target reliability as the baseline, at least in general, takes into account the uncertainty in the resistance. Therefore, the most significant effect on the load factor needed to address the uncertainty in the downdrag load estimate is the uncertainty in the method used to estimate downdrag.

Based on the calibration results, it is recommended that the load factors provided in Table 12 be used for downdrag. These recommended load factors reflect an approximate average value for the various cases evaluated. For a more refined analysis, the designer may wish to select a load factor that more specifically reflects the DD/DL ratio for the design case being considered.

As discussed in Section 4, an update to the Reese and O'Neill (1988) Method (i.e., O'Neill and Reese, 1999) has been proposed for drilled shaft design. For relatively low shear strength clays, the original method will produce the same unit skin friction values relative to the updated method. Since O'Neill and Reese (1999) will produce the same skin friction values as the 1988 method shown in Table 12, the load factor provided in Table 12 for shafts is applicable to the updated shaft design method.

Note that there is really no minimum load factor to be used for the case where the load acts as a restoring force. For the uplift limit state, the negative skin friction simply functions as additional resistance in uplift, and uplift resistance factors should be applied to the skin friction resistance in the downdrag zone.

**Table 11. Summary of calibration results for downdrag of piles and shafts.**

| Description of Limit State and Conditions | Calculation Method Used as Basis for Statistics Selected | Bias | COV | Resistance Factor Used | DD Load Factor needed to Match φ Obtained without DD Loads (DD/DL = 0) | DD Load Factor needed to Match φ Obtained without DD Loads (DD/DL = 0.5) | DD Load Factor needed to Match φ Obtained without DD Loads (DD/DL = 1.0) |
|---|---|---|---|---|---|---|---|
| Concrete Piles Bearing in Sand with Downdrag in Clay, using α-Tomlinson Method for DD | Resistance in Sand - Nordlund | 1.02 | 0.48 | 0.40 | 1.0 | 1.2 | 1.3 |
| | DD - α-Tomlinson (using compression stats) | 0.87 | 0.48 | | | | |
| | Resistance - CAPWAP BOR | 1.16 | 0.34 | 0.65 | 1.0 | 1.2 | 1.35 |
| | DD - α-Tomlinson (using compression stats) | 0.87 | 0.48 | | | | |
| Steel Pipe Piles Bearing in Sand with Downdrag in Clay, using α-Tomlinson Method for DD | Resistance in Sand - Nordlund | 1.48 | 0.52 | 0.53 | 1.0 | 1.0 | 1.0 |
| | DD - α-Tomlinson (using compression stats) | 0.64 | 0.50 | | | | |
| | Resistance in Sand - Nordlund | 1.48 | 0.52 | 0.53 | 1.0 | 1.2 | 1.3 |
| | DD - α-Tomlinson (using uplift stats) | 0.95 | 0.57 | | | | |
| | Resistance – CAPWAP BOR | 1.16 | 0.34 | 0.65 | 1.0 | 1.0 | 1.1 |
| | DD -α-Tomlinson (using compression stats) | 0.64 | 0.50 | | | | |
| | Resistance – CAPWAP BOR | 1.16 | 0.34 | 0.65 | 1.0 | 1.4 | 1.65 |
| | DD - α-Tomlinson (using uplift stats) | 0.95 | 0.57 | | | | |
| | Resistance – FHWA Gates EOD | 1.07 | 0.53 | 0.38 | 1.0 | 1.0 | 1.0 |
| | DD - α-Tomlinson (using compression stats) | 0.64 | 0.50 | | | | |
| | Resistance – FHWA Gates EOD | 1.07 | 0.53 | 0.38 | 1.0 | 1.4 | 1.6 |
| | DD - α-Tomlinson (using uplift stats) | 0.95 | 0.57 | | | | |

| Description of Limit State and Conditions | Calculation Method Used as Basis for Statistics Selected | Bias | COV | Resistance Factor Used | DD Load Factor needed to Match φ Obtained without DD Loads (DD/DL = 0) | DD Load Factor needed to Match φ Obtained without DD Loads (DD/DL = 0.5) | DD Load Factor needed to Match φ Obtained without DD Loads (DD/DL = 1.0) |
|---|---|---|---|---|---|---|---|
| Concrete Piles Bearing in Sand with Downdrag in Clay, using λ-Method for DD | Resistance in Sand - Nordlund | 1.02 | 0.48 | 0.40 | 1.0 | 1.0 | 1.0 |
| | DD - λ-Method (using compression stats) | 0.76 | 0.29 | | | | |
| | Resistance - CAPWAP BOR | 1.16 | 0.34 | 0.65 | 1.0 | 1.0 | 1.0 |
| | DD – λ Method (using compression stats) | 0.76 | 0.29 | | | | |
| Steel Pipe Piles Bearing in Sand with Downdrag in Clay, using λ-Method for DD | Resistance in Sand - Nordlund | 1.48 | 0.52 | 0.53 | 1.0 | 1.0 | 1.0 |
| | DD - λ-Method (using compression stats) | 0.67 | 0.55 | | | | |
| | Resistance in Sand - Nordlund | 1.48 | 0.52 | 0.53 | 1.0 | 1.0 | 1.1 |
| | DD - λ-Method (using uplift stats) | 0.72 | 0.52 | | | | |
| | Resistance – CAPWAP BOR | 1.16 | 0.34 | 0.65 | 1.0 | 1.0 | 1.1 |
| | DD – λ Method (using compression stats) | 0.76 | 0.29 | | | | |
| | Resistance – CAPWAP BOR | 1.16 | 0.34 | 0.65 | 1.0 | 1.0 | 1.1 |
| | DD – λ Method (using uplift stats) | 0.72 | 0.52 | | | | |
| Shafts Bearing in Sand, using Mixed Construction Methods, with Downdrag in Clay | Resistance - Reese and O'Neill (1988) for Sand | 1.71 | 0.60 | 0.35 | 1.0 | 1.2 | 1.3 |
| | Reese and O'Neill (1988) for clay (using skin friction only stats) | 0.87 | 0.37 | | | | |
| Shafts Bearing in Mixed Soils, using Mixed Construction Methods, with Downdrag in Clay | Resistance - Reese and O'Neill (1988) for Sand | 1.19 | 0.30 | 0.57 | 1.0 | 1.15 | 1.25 |
| | Reese and O'Neill (1988) for clay (using skin friction only stats) | 0.87 | 0.37 | | | | |

**Table 12. Recommended load factors for downdrag loads, strength limit state.**

| Method Used to Estimate Downdrag Loads | Load Factor |
|---|---|
| Piles, α-Tomlinson Method | 1.4 |
| Piles, λ-Method | 1.05 |
| Shafts, FHWA Method (Reese and O'Neill, 1988) | 1.25 |

# 8. Conclusions

The development history and logic regarding the resistance factors, and downdrag load factor, needed for foundation design are presented. In general, estimation of geotechnical resistance, and loads, can involve considerable engineering judgment, and this judgment is difficult to quantify statistically. Furthermore, the available data upon which the needed input statistics were based were limited in terms of quantity and quality in some cases. In other cases, considerable high quality data were available to develop the needed statistics for reliability theory input. Therefore, the final selection of resistance factors considered both the statistical reliability of the method and the level of safety implied by past successful design practice. The degree of reliance on past successful design practice depended upon the quality of the database available, and how well that data modeled reality. In addition, the calibration studies summarized herein have been conducted over a considerable period of time, and changes and upgrades in design procedures have occurred since those studies were conducted. Judgments were made regarding how the available calibration results apply to the newer design methods.

It must be recognized that there is some inherent reliability in past successful design practice and the safety factors used in those design practices. Therefore, if it is decided to deviate from those past successful design practices, there must be a strong reason for doing it. A large and reliable database coupled with reliability theory could be an adequate reason for deviating from past practice, especially if it is recognized that past practice has been excessively conservative, or if past practice has resulted in a higher than acceptable failure rate. This philosophy has been used in the establishment of recommended resistance factors for design in previous calibration work. The recommended resistance factors provided herein were developed consistent with that philosophy.

Finally, the calibration results reported herein have only considered the factors used to quantify level of safety (e.g., safety factors, load factors, statistical data representing the uncertainty in the loads and resistance, etc.). Trial comparative designs between ASD and LRFD using the recommended resistance factors have not been conducted. Such trial comparative designs should be conducted to increase confidence in the recommended resistance factors.

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